# BRICK ENGINEERING

## HANDBOOK OF DESIGN



# Principles

*of*

# BRICK ENGINEERING

## Handbook of Design

by

HARRY C. PLUMMER

*Director of Engineering and Research*
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and

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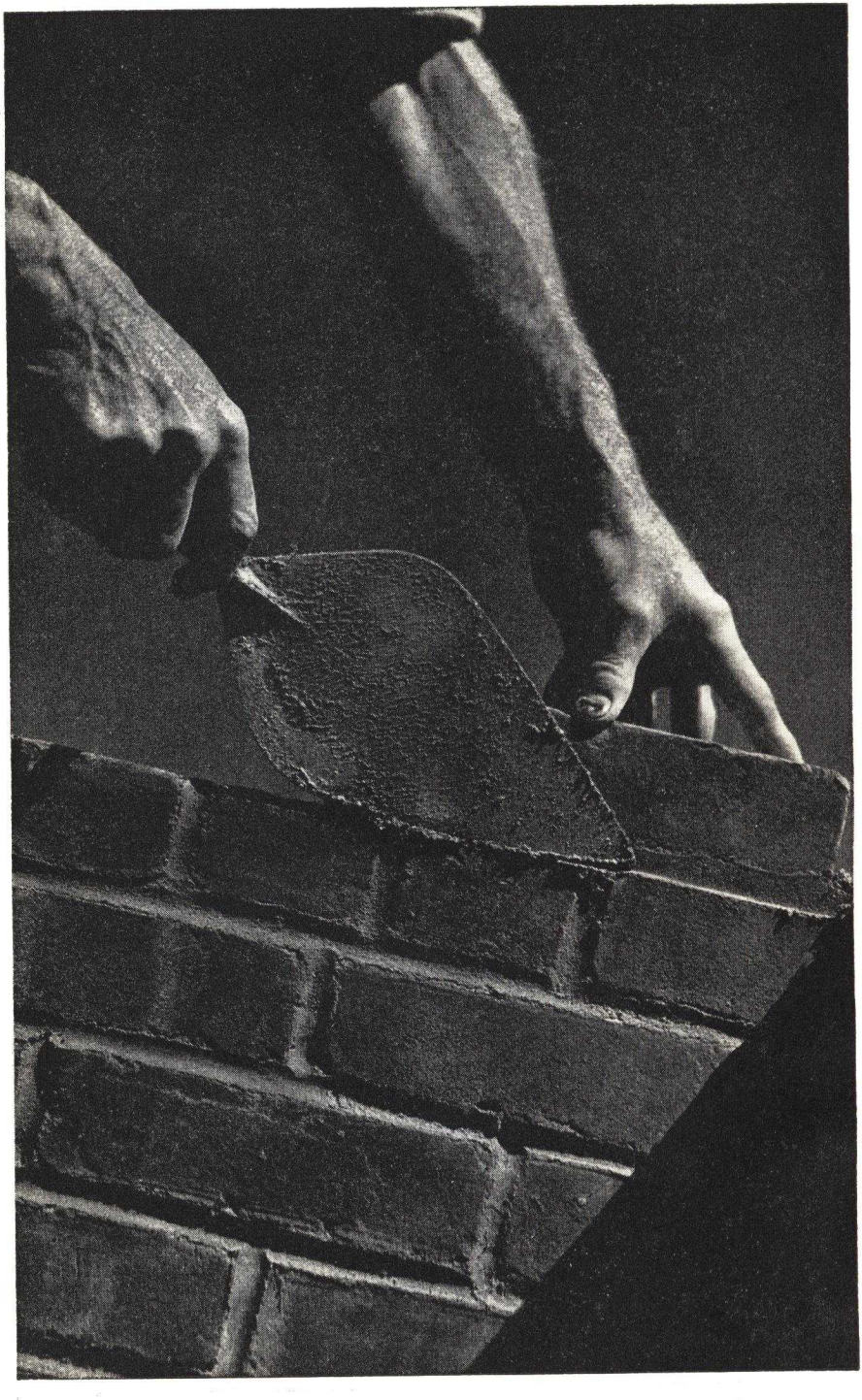
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**Price $4.00**

Published by
**Structural Clay Products Institute**
Washington, D. C.

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# PREFACE

BRICK is one of the oldest of our modern building materials. It was used extensively in many parts of the world long before the present methods of structural design and stress analysis were developed and, consequently, many common practices in design and construction methods are the results of long usage rather than of theoretical analyses. In ancient times, much of this knowledge was passed on from one generation of craftsmen to the next by word of mouth, and this may be the reason that the volume of literature devoted to brick masonry is small as compared to the writings on comparable types of construction.

During the past two decades particularly, extensive research has been conducted on plain and reinforced brick masonry and while the results of the various investigations are available individually in printed form, no attempt has been made in recent years to correlate this work and to include in one volume the best available information regarding brick and brick masonry. The need for such a book has been expressed by many in the building industry and it was in recognition of this demand that the authors undertook the preparation of this volume.

While no effort has been spared to obtain complete and accurate data on the various subjects covered, it is improbable that this first edition will be entirely free from error. The authors may have also overlooked some important data which should have been included, but such omissions were not intentional. Notice of any such errors or omissions or other constructive criticism will be welcomed.

In the preparation of this book, liberal use has been made of the publications of the former Brick Manufacturers Association of America, American Face Brick Association and Structural Tile Association, all now merged with the Structural Clay Products Institute. Wherever possible, the original source of data contained herein is given in the text with proper acknowledgement of the author. In addition to the above, however, the authors wish to express particular appreciation for the valued suggestions and criticisms submitted by Mr. D. E. Parsons, Chief, Masonry Section, National Bureau of Standards, Mr. J. W. McBurney, Senior Technologist, National Bureau of Standards, Mr. F. O. Anderegg, Building Consultant and Mr. Hugo Filippi, former Consulting Engineer, author of "Reinforced Brick Masonry" and now superintendent of the Illinois Brick Co. Acknowledgement is also made of the assistance given by the staff of Structural Clay Products Institute.

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WASHINGTON, D. C.
*September, 1939.*

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# BRICK

## *An Appreciation*

**by**

**HENRY H. SAYLOR**

**Associate Editor Architectural Forum**

Reprinted from American Architect and Architecture July 1936

ONE meets quite frequently, in these days, the man who insists that building for modern needs means complete abandonment of old methods and old materials. He is likely to argue that we should never again build a stone house of the Cotswold style — it was for another day and another mode of life. Indeed, if encouraged — or perhaps even without encouragement — he will probably go so far as to say that, with our knowledge and our machinery, we should never again pile stone on stone to enclose space; it would be so much easier, quicker, and less expensive to put up two thin sheets of steel, with proper insulation between them. Then, warming to his subject, he will launch the most scornful of his shafts — even though it is somewhat dulled by too frequent use: "Why, we still lay brick on brick to make a wall, just as the Babylonians did before Christ was born!"

Doubtless it would serve to fan our inconoclast into a mute rage if I ventured the opinion that, unless mankind loses or deliberately abandons his search for beauty, we shall be laying brick on brick some thousands of years in the future. If, as someone has well said, all the past is but a prelude to the future, I think I shall have little difficulty in establishing the likelihood of that contention.

From a sun-dried clay tablet made in the time of Sargon of Akkad, founder of the Chaldean Empire, nearly four thousand years before Christ, we have evidence that man patted the clay of his river valleys into cakes, dried them in the sun, and used them for building, some thousands of years before the earliest recorded history.

It is quite likely that some progressive of that dim past wagered with his fellows that some material far better than clay would soon supplant it. Nevertheless, when the Tower of Babel came to be built, it was brick that made it — brick which by that time was burned hard by fire rather than merely dried in the sun. Still later, coming down to the time of Nebuchadnezzar (604-561 B.C.), man was still laying brick on brick to build his walls, but by that time he knew not only how to burn it but how to enamel it as well.

Even in that treasure house of achievement in stone, Egypt, there is one pyramid at Dashur, just south of Cairo, that is of brick, and an inscription boasts of it in words roughly translated thus: "Disparage me not by comparing me with pyramids built of stone. I am as much superior to them as Ammon is superior to the rest of the deities. I am constructed of brick made from mud which adhered to the ends of poles and was drawn up from the bottom of the lake."

The ancient and honorable art of brick making seems to have originated on the Mesopotamian plains. Eastward it spread to Persia, India, China. Westward it spread to Egypt, Asia Minor, Greece, Rome, and from Rome to Europe and the Western World.

In this long march onward through the centuries, there is one point that should be emphasized. The brickwork of Rome was something more than the brickwork of Babylon; the brickwork of Mediaeval England was something far more than that of Rome. After Assyria and Egypt had done what they could with brick, Rome borrowed it and built the Baths of Caracalla and the Basilica of Constantine. After Rome, Spain carried the art forward and built the Mosque of Cordova. France lifted higher the torch, with her chateaux, England with her Hampton Court, Holland with her Guild Halls.

When one country after another found brick as used by a neighbor, it found something more than a piece of clay; it found an art. How that art was borrowed, developed to greater and greater heights, and passed on to other men and other lands for further development, is one of the great sagas of architecture.

Take, for example, the story of English brickwork. It was the Roman builders in England who made and used the first brick in a glorious history. These brick were what we should call tiles — made of clay beaten flat, dried on the ground, stacked on edge in kilns, and burned by wood fires. Naturally, they varied widely in length, breadth and thickness. Sometimes they were triangular in shape, with the best flat edge on the face of the wall, the interior of which was a most heterogeneous conglomerate. Incidentally, these Roman brick were reclaimed, cleaned and used again in later English work.

The making of brick as an industry appeared in England in the Thirteenth Century, but it was not until the days of Henry VIII (1509-1547) that the craft reached a high degree of excellence — probably attained largely through Flemish influence. Then came the Fire of 1666, and London became a town of brick instead of a town of wood. And even then the art of brick masonry in England was in its infancy. Its full stature was not reached until, after long development through the reigns of Queen Anne and the Georges, it achieved the grand old manor houses of the Eighteenth Century.

Lest we establish brick masonry in our minds as merely a variation in wall texture, let it be recalled that its small units and ease of handling probably brought about the discovery of the arch and the vault. Inevitably these would have been discovered in time by the stone masons, but since Assyria seems to have been the first home of the arch, and the Assyrians were brickbuilders, we shall have to credit the clay product at least with an assist.

Brick, it should be remembered, is also one of the very few building materials that have brought into being a distinctive style. Stone, of course, has achieved this, also wood, and in our own day steel has joined the ranks of those fundamental materials, without which man could not have built certain of his creations. If you should doubt that architectural style has been achieved through the use of brick — a style in which no other form of masonry would have sufficed — consider The Midi of Southern France, centering about Toulouse and Albi. Possibly brick has been used here with greater skill and more taste than anywhere else in the world. Dean William Emerson and Georges Gromort think so, and their recent book, "The Use of Brick in French Architecture," is a document in evidence. Again, is it possible to conceive of some of the Dutch work executed in any material other than brick? Brick in The Midi transcends the merely utilitarian role and achieves — as in the Cathedral of Albi — an architectural quality that could not have been attained in marble or stone. In passing, it is worth noting that the individual brick used in this work are not often over an inch and a half thick, but they are from fifteen to seventeen inches long, and in depth are occasionally eleven inches. The joints in the old work average about three-quarters of an inch, and sometimes reach one and five-eighths.

Speaking of brick sizes, man has never fully made up his mind as to what is the ideal — and the reason, probably, is that what is best for one purpose is not best for another. The dome of Santa Sophia in Constantinople is built of brick that are twenty-four to twenty-seven inches square and two inches thick. The mortar joints are almost the thickness of the bricks. In this instance the brick serves a purely utilitarian purpose, for it is sheathed on the exterior with lead and on the inside with glass mosaic.

To Vitruvius, just before the beginning of the Christian Era, a brick was about twelve by eighteen inches, and little thicker than a tile. He tells us also that the Greeks measured their bricks by the palm of the hand — the pentadoron was five palms square, the tetradoron four palms square.

In this matter of size, our familiar mentor, Batty Langley, had at least the germ of the modular idea in his head when he wrote (in 1749) of bricks two and a half inches thick "to rise in four courses to a foot in height."

Experiments looking toward the achievement of a pattern in brickwork must have been initiated by very early craftsmen, but apparently the progress was slow. What is called Old English Bond (alternate courses of headers and stretchers) had come into general use in England in the Fifteenth Century, and it was used almost exclusively until Flemish Bond was introduced about 1625-50. Meanwhile the French were making more rapid progress. What are now called English Cross Bond and Dutch Bond were used by the Mediaeval brick masons in addition to the two bonds mentioned. These masons also developed the scheme of diaper patterns, using vitrified headers or brick of another color — purples, blues, grays, and sometimes almost black. The scheme was welcomed by the French particularly for its aid in lending scale to walls of the larger buildings. Like most aids to the designer, however, it was overdone, and finally reached its limit in the tour de force of a dovecote for Boos Manor, Rouen. Here the pattern became so marked and intricate, so full of sharply defined panels, as to break the fundamental laws of proper structural bonding. Softened by time, the dovecote now offers a beauty that it could hardly have possessed when new.

Nor were the brick masons satisfied when they had achieved beauty of texture and color in a wall surface. The third dimension called to them, and they succeeded in getting away from the plane by molding special forms, by grinding ordinary brick to a profile, and by carving rectilinear brickwork after it had set. All three methods were employed from early times.

Naturally, our own beginnings in America took as a point of departure the art as it was then practiced in England and in the Low Countries. Adobe brickwork had been found by the Spaniards in Peru and Mexico, and they developed it in their settlements in our own Southwest. The industry which stems from English and Dutch practice, however, took root in Virginia in 1611, and in Massachusetts in 1629. While the colonies along the North Atlantic were doing an honest, craftsmanlike job of putting up a few brick buildings among their far more numerous structures of wood, the planters farther south, particularly in Maryland and Virginia, were aspiring to the higher reaches of the art.

Brick is perhaps the most widely adaptable of our wall materials. For the poor man it will accept the humblest utilitarian role. From that it responds, all the way up the scale, to every demand made for higher degrees of quality in materials, workmanship and design, lacking no merits that may be required of a material in the class of unlimited cost. It does not compete with stone or marble for the designer's favor, nor does it brook competition from them. At its best, it stands alone, no more to be compared with other materials than an iris can be compared with a peony. Each is comparable only with other members of its own family.

# CHAPTER 1
# BRICK

## 101. DEFINITION OF BRICK

The term, brick, has been used to designate a building unit of burned clay or shale. In recent years, other materials used in producing a building unit of approximate brick size have caused some misunderstanding of the meaning of the term.

The Committee on Nomenclature and Definitions (E-8) of the American Society for Testing Materials has published the following definition of brick:

> "**Brick** — In the case of structural and road building material, a small unit, solid or practically so, commonly in the form of a rectangular prism, formed from inorganic, non-metallic substances and hardened in its finished shape by heat or chemical action.
>
> "NOTE — The term is also used collectively for a number of such units, as 'a carload of brick.'
>
> "In the present state of the art, the term brick, when used without a qualifying adjective, should be understood to mean such a unit, or a collection of such units, made from clay or shale hardened by heat. When other substances are used, the term brick should be suitably qualified unless specifically indicated by the context."

It is, therefore, officially recognized that unless suitably qualified, a brick is a unit of burned clay or shale. Of course, architects, engineers, contractors, building officials and authoritative dictionaries have always considered a brick to be a unit of burned clay or shale.

## 102. THE RAW MATERIAL

Clays and shales from which brick are made are found in many locations all over the world. Nature has, through centuries of time, produced a raw material which might properly be called a mixture of stable chemical compounds. Clay differs from shale, not so much in chemical composition as in physical structure.

Clays (surface clays, fire clays or semi-clays) are derived from the weathering of silicate rocks, containing sufficient hydrous silicate of alumina to produce a material which has plastic qualities.

Shales may be defined as moderately hard clay strata which have been compressed, sometimes heated, but not to the point of chemical alteration, and possessing a structure with natural planes of cleavage, but not definite enough to split like slate.

Natural clays and shales vary in their chemical composition in various parts of the country. This chemical composition of clays and shales largely determines the color of bricks and along with other physical characteristics often exerts an influence on the process of manufacture.

## 103. MANUFACTURING METHODS

There are a number of different processes involved in the manufacture of brick such as:

(a) Selection or choice of materials.
(b) Winning or obtaining the clay or shale.
(c) Weathering, usually by exposed storage.
(d) Grinding, disintegrating, cleaning and sorting.
(e) Proportioning of materials.
(f) Sizing or screening.
(g) Mixing and tempering; developing plasticity and uniformity.
(h) Shaping.
(i) Drying.
(j) Burning.

In the following discussion, the term, clay, is used in a comprehensive manner to include all types of clays and shales used in the manufacture of brick.

An important factor in the selection of a clay is its ability to produce the desired quality of brick in an efficient and economical manner. Clays are usually removed by surface digging, quarrying or mining, depending upon the location and nature of the deposit. Surface digging may be done by any number of methods, including the use of power shovels and shale planers. Quarries are frequently worked in benches similar to the method used in stone quarries. Mining is justified if the clay is high grade. All methods of winning the clays require utmost consideration in the conservation of material, drainage problems and transportation of materials to the storage piles or bins.

The preparation of clays embraces a threefold object: (1) the homogenizing of the materials, (2) the purification of the clays and (3) the development of plasticity. If the processes of mixing involve simply the use of one material of practically constant physical and chemical constitution, the problem becomes simple. But where it involves the mixing of dissimilar substances, such as two different clays, or shale and clay, or clay and grog (finely-ground, burnt clay), it is important that the materials be thoroughly mixed in order to obtain a uniform product.

The preparation of clays herein described is somewhat typical of most processes. From the storage piles or bins, the clay is usually conveyed to the granulator, a machine consisting essentially of a semi-cylindrical tank containing a revolving steel shaft equipped with knives to break up the chunks of clay. The knives are pitched and arranged so as to act as a screw conveyor, moving the mixed and granulated material toward the discharge end.

The clay is now ready to be ground. If it happens to contain large stone or hard lumps of clay, it may be put through conical rolls which eliminate the stones and crush the lumps of clay. The grinding may be done in any one of many types of grinders. Vibrating screens may be used to further eliminate large particles.

After the clay has gone through these preliminary stages of preparation, it is necessary to prepare it further by kneading or pugging it, into a homogeneous plastic mass. This is called, tempering, which is usually accomplished in a pug mill, consisting of a chamber containing one or more revolving shafts with blades. Plasticity is produced by thoroughly mixing the material, adding water, if necessary.

In forming or molding brick, there are generally three processes, i. e. "soft-mud", "stiff-mud", and "dry press". The preliminary mixing processes are fundamentally the same, except that the water content varies from considerable in the soft-mud to practically none in the dry-press process.

In the soft-mud process, the prepared clay is mixed wet to make a soft mud or paste with a water content of 20 to 30 per cent and formed by light pressure in a mold. Where this process is used, the clay is usually found wet in the ground.

There are two types of soft-mud brick; sand-struck and water-struck. When the insides of the molds are sanded to prevent the clay sticking, the product is known as sand-struck brick, sometimes called sand-moulded. When sticking of the clay is prevented by wetting the molds with water, the process is called slop molding, and the product is known as water-struck brick. The soft-mud brick machines perform essentially the same operations and produce a product much like hand-molding, which method of making brick goes back to remote antiquity.

In the stiff-mud process, the clay is kept just wet enough to stick together when worked up in a mass. The moisture content of the clay used in the stiff-mud process ranges from 12 to 15%. The clay is forced by means of an auger or screw through a die, much as tooth paste is pushed out of the tubes. When the die opening is such that a column of clay 2¼ x 3¾ inches is formed and the units are formed by cutting the column into eight-inch lengths, the bricks are said to be "end cut." When the column is 3¾ x 8 inches and is cut at 2¼-inch intervals, the bricks so formed are "side cut."

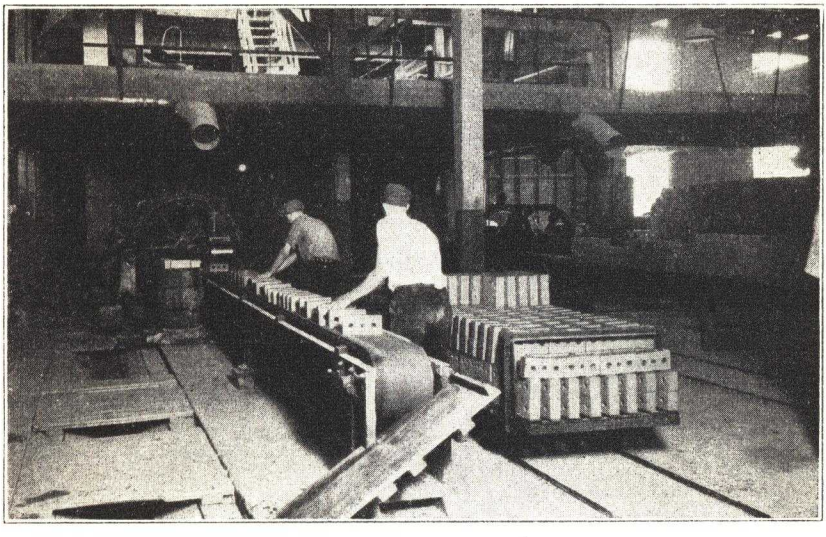


*Figure 1*
*Stiff mud brick after passing through die and cutter and being stacked on cars preparatory to drying.*

A recent and important development in the stiff-mud process, called de-airing, is accomplished by a vacuum chamber attached to the auger machine. Varying degrees of vacuum are used, ranging from less than half to practically a complete vacuum (14 to 29 inches of mercury). The advantages claimed for de-airing are increased workability and plasticity, and strength in the "green" (undried) brick. It has been found that clays respond differently to de-airing. Its effects on many clays have been very favorable, with substantial improvements in the finished ware; while on a few clays it has been found unfavorable and the process discontinued. Much will undoubtedly be learned about the use and effects of de-airing in the next few years.

In the dry-press process, a much smaller amount of water (5 to 7%) is used. The clay is relatively non-plastic and the brick unit is formed by high pressures, sometimes 6000 or 7000 pounds per square inch.

Before burning, it is necessary to dry the "green" brick. The methods used range from open air drying (now nearly obsolete) to elaborate constructions where temperature, humidity and velocity of air are carefully controlled. The objects sought in drying are to remove some of the water from the brick, previous to burning, so that time and fuel will be saved in the burning process and to increase the mechanical strength of the green brick, so they can be set to a greater height in the kiln without injury. The drying process must not be too rapid and usually requires about three days.

The next step is the burning process, which is undoubtedly the most important one in the manufacture, and requires from 60 to 100 hours. It consists of several stages, known as water smoking, dehydration, oxidation, vitrification and cooling.

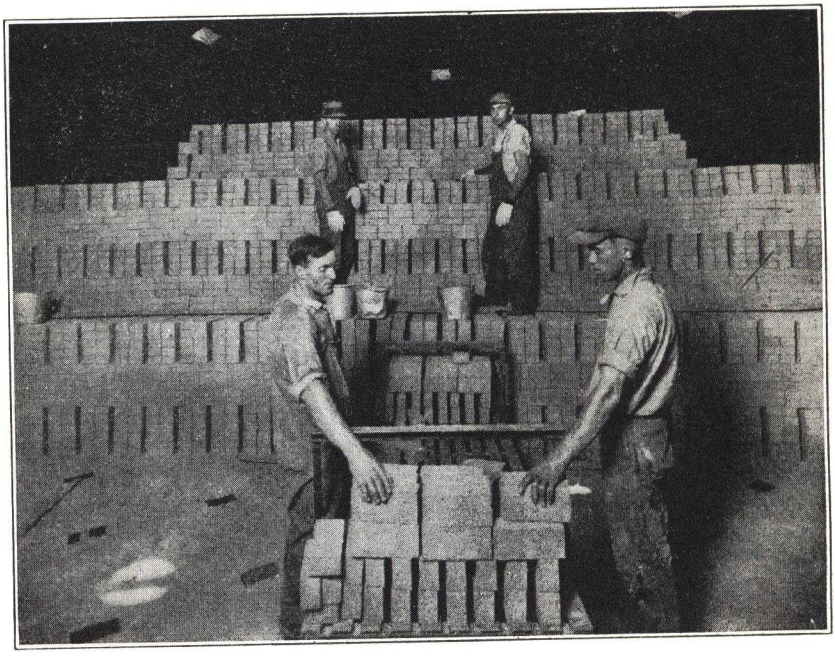


*Figure 2*
*Brick being set in a periodic type of kiln.*

The whole burning process is carried on in kilns, which are described later. The brick are "stacked" in a manner to permit the hot gases to flow freely through the entire mass and raise the temperature of the units uniformly.

During the water-smoking period, which lasts about 12 hours, the temperature is raised slowly to approximately 300° F. The object of this period is to expel the remaining free water which had not been removed in "drying."

Following the water-smoking period, the kiln temperature is gradually raised to the dehydration stage of the "burn," which starts at about 800° F to drive the chemically combined water out of the brick. Dehydration is practically complete at 1200 or 1300° F.

Oxidation is started during the dehydration period and is usually complete at 1400 to 1700° F. An even temperature is required while plenty of air is introduced.

During this period, all combustible matter is oxidized or burned off, ferrous iron, if any, is oxidized to the ferric condition, carbon dioxide is driven off and sulphur is expelled from the pyrites (iron disulphide).

From the oxidation period, the heat is steadily increased to the vitrification period. There are two stages of vitrification — incipient and complete.

Incipient vitrification is that stage when the clay is heated until it "flows" just enough to make the grains fuse together, but not enough to prevent recognition of the smaller grains, nor to close all the pores. Most building brick are burned to the incipient stage (1500 to 2100° F).

Complete vitrification is that additional stage when the temperature of the clay is sufficient to fuse all the grains and close all the pores and render the mass impervious. Paving brick are burned to practically complete vitrification.

"Flashing," while not necessarily a stage of burning, is a feature toward the end of the burn. It is done by reducing the fire, which produces certain colors or shades of colors, varying with different types of clays.

Cooling after burning is very important and must be done gradually to avoid checking or cracking of the brick surface. It also affects the color of the brick. Two or three days are usually required for cooling.

Bricks are "burned" in a kiln which is an oven chamber or series of such chambers. Kilns are of two general types, periodic and continuous, and may be classified as follows:

**1. Periodic Type**

A. Scove kiln.
B. Clamp kiln (permanent walls).
C. Round or rectangular up-draft kilns (permanent walls and crown).
D. Round or rectangular down-draft kilns (permanent walls and crown).

**2. Continuous Kilns**

A. Moving Fire Zone Type

a. Chamber type composed of a series of chambers with inter-connecting ports in the common side-walls. A longitudinal flue below the surface of the kiln floor usually runs the entire length of the chamber kiln. If the kiln is built in a circle or oval this flue may not be present.

b. Continuous kilns similar to "a" but without permanent division walls. The chambers are formed by paper diaphragms extending from wall to wall and from floor to crown.

B. Fixed Fire Zone Type

a. "Straight-line" car tunnel kiln, in which the bricks are loaded in the dry state on cars and introduced into the tunnel kiln, receiving their heat treatment as they approach the fixed firing zone and their maturing temperature in the firing zone. From whence, in their progress through the discharging end of the kiln, the brick are slowly cooled.

b. Moving "circular hearth" tunnel kiln. The firing methods of this are the same as "straight-line" car tunnel kiln, except that the circular hearth to all intents and purposes is continuous.

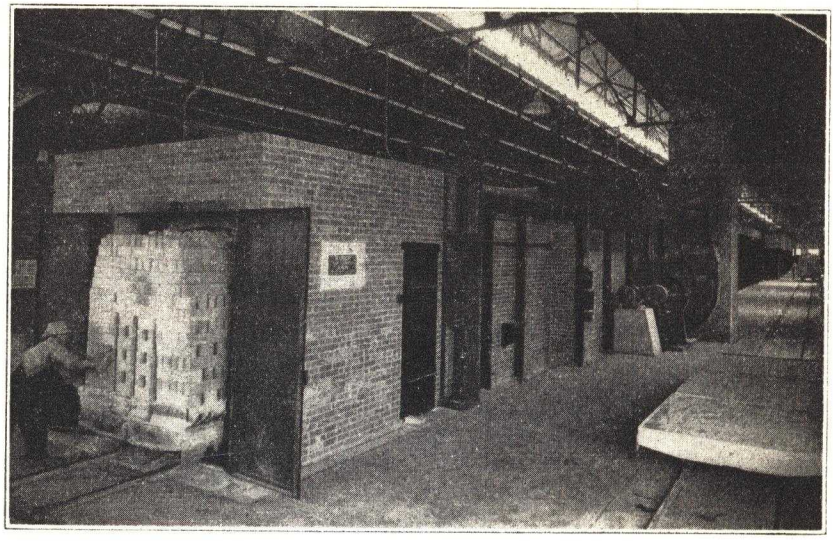


*Figure 3*
*Car of brick entering continuous type (tunnel) kiln.*

The earliest type of brick kiln is the scove kiln, which is merely a pile of dried bricks set with parallel cross tunnels at the bottom of the setting. These tunnels are about 4 feet apart, fires are built in the tunnels and the heat passes up through the mass of the setting. The heat is retained in the setting sufficiently to fire the brick, by plastering the outside of the setting with a mixture of clay, sand and water, so as to form a crust. The same thing is accomplished on top of the setting by placing the bricks close together and venting them as needed to provide circulation to pull the heat up through the setting. In fact the word "Scove" in its ancient sense means "crust."

A clamp kiln is merely a scove kiln where the scove is affected by means of permanent walls, forming a hollow box open at the top, with small openings at the bottom to accommodate the cross firing tunnels.

The next group of up-draft and down-draft kilns is merely another step in the development of the scove or clamp kilns as the art grew older, and the descriptive term indicates the direction that the heat flows.

Following this, the moving fire zone type of continuous kiln, was accomplished by grouping together a number of periodic kilns with inter-connecting passages, thereby permitting the fire to move from one kiln to another.

The most modern development is the fixed fire zone type, or tunnel kiln, with continuously moving platform or cars loaded with brick. This system lends itself well to the mechanization of brick plants, thus conforming with standard practice in modern industry. The tunnel kiln may be built either straight or circular.

The use of hydraulic propulsion is an important part of this development, as it permits a great tonnage of brick to be conveyed through the various operations with an inconsiderable amount of power.

The fixed fire zone, or tunnel type of kiln, is extremely flexible in its application and millions of bricks are now burned annually by this method. Advantages claimed for the tunnel kiln are more accurate and efficient control of the heat and a more uniformly burned brick.

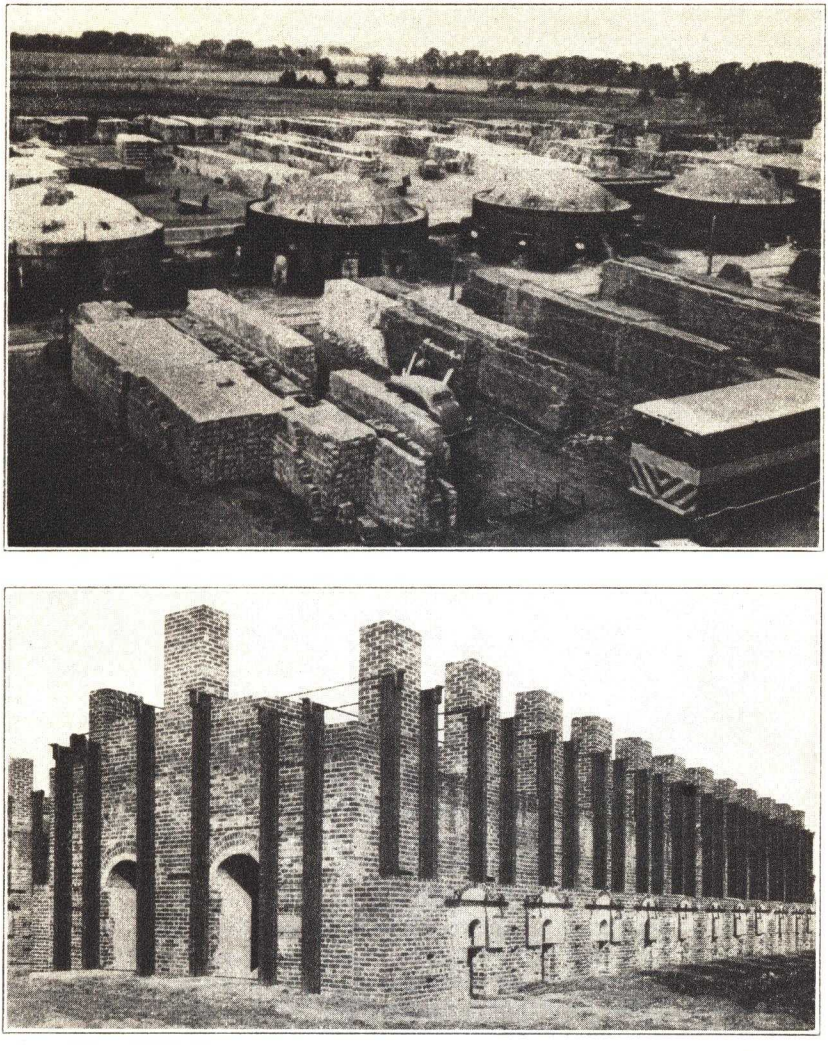


*Figure 4*
*Two types of periodic kilns. Round (bee-hive) kilns shown in upper view. Lower view shows clamp kiln.*

## 104. PROPERTIES OF BRICKS

The sections which follow summarize the information available on various properties of brick units. Among the properties considered are shape, size, appearance, and those properties depending upon capillary structure, such as porosity and permeability; several strength measures such as compressive, transverse, tensile strength and shearing strength; such measures of hardness as resistance to abrasion and shock, and miscellaneous properties including elasticity, thermal conductivity, thermal expansion, expansion due to moisture content and in addition, certain chemical properties such as soluble salt content with its effect on efflorescence, etc.

## 105. SHAPE

Brick is defined as a rectangular unit, usually solid or nearly so. Side cut brick are frequently made with holes through the flat of the brick for the purpose of saving weight and improving the burning. It is also claimed that the mortar bond is improved by the key action in these holes. Soft-mud and some dry-press bricks are frequently formed with a shallow depression or frog in one of the flat faces. However, building codes usually provide that if the net volume of a brick is at least 75% of the gross volume, it is regarded as a solid unit. Various special shapes are molded or ground, such as radial brick for chimneys, compass brick for arches, ornamental shapes, angle brick, bullnose brick, beaded brick, jack arches, sills, etc.

## 106. SIZE

In the United States, the American Society for Testing Materials has standardized the size of building brick as follows:

"The standard sizes shall conform to the following table of dimensions with a permissible variation, plus or minus, of $\frac{1}{16}$ inch in depth, $\frac{1}{8}$ inch in width and $\frac{1}{4}$ inch in length."

| Type | Depth Inches | Width Inches | Length Inches |
|---|---|---|---|
| Common brick | 2¼ | 3¾ | 8 |
| Rough-face brick | 2¼ | 3¾ | 8 |
| Smooth-face brick | 2¼ | 3⅞ | 8 |

Federal Specifications for Brick; Building (Common), Clay, designated as Federal Specifications Board Specification SS-B-656, adopted June 28, 1932, has the following comments on size:

> "D-1. The size of brick shall be as specified in the invitation for bids with permissible maximum variations from the specified size, over or under, of one-eighth inch in breadth or depth, and one-quarter inch in length. (See I-1a for sizes.)
>
> "I-1a. The standard size of brick, as established by simplified practice, is 2¼ x 3¾ x 8 inches. At the time this specification is prepared this size is not obtainable in some parts of the country, and purchasing officers should ascertain the size required."

There are several special sizes of rectangular brick made in various sections of the country. Some are: double brick, approximately 5″ x 3¾″ x 8″, which is two brick high; dubrick, 2¼″ x 8″ x 8″, which is two brick wide. Others might be called 1½ brick, either in height (4″ x 3¾″ x 8″) or length (2¼″ x 3¾″ x 11¾″).

Other special sizes of brick are the so-called Groutlock and adobe brick, which are being used extensively in Southern California. The Groutlock brick has a regular size of 3″ x 3½″ x 9″, and the adobe size is 3″ x 3½″ x 11¾″. Groutlock brick usually have one or more beveled edges. Many special shapes and sizes are made for use in reinforced brick masonry construction or specialized installations.

As a result of a survey made in 1930 covering approximately 50% of the building brick produced in the United States, it can be stated that one-third of the production sampled falls within the tolerances for standard size specified in the A.S.T.M. specification. These data were published in INDUSTRIAL STANDARDIZATION, January, 1935.

Table 1 is from a table of brick sizes and weights translated from an article in German by G. Paschte, "The Human Hand and Dimensions of Brick." This table was originally in the metric system and has been converted into English units. It shows the sizes and weights of bricks produced in the principal countries of the world as well as certain medieval and ancient bricks.

## TABLE 1

## SIZES AND WEIGHTS OF BRICK PRODUCED IN THE PRINCIPAL COUNTRIES OF THE WORLD

| Name of form of brick Country or Community of Occurrence | Dimensions in Inches | | | Vol. in Cu. In. | Vol. ratio to U.S. Stand. | Weight in Lbs. |
|---|---|---|---|---|---|---|
| | Length | Breadth | Thickness | | | |
| | 1 | 2 | 3 | 4 | 5 | 6 |
| Belgian | 4.55 | 3.35 | 1.75 | 27.05 | 0.400 | 1.85 |
| Holland, Issel | 6.30 | 3.15 | 1.55 | 31.25 | 0.465 | 2.10 |
| Hamburg, Geest | 7.10 | 3.40 | 1.80 | 43.45 | 0.645 | 2.95 |
| Flanders | 7.10 | 3.35 | 1.95 | 46.70 | 0.690 | 3.20 |
| Holland, Benlosche | 7.10 | 3.45 | 1.95 | 47.75 | 0.705 | 3.25 |
| American (1913) | 7.90 | 3.95 | 1.95 | 61.00 | 0.905 | 4.15 |
| Paris (small size) | 8.65 | 3.95 | 1.95 | 67.10 | 0.995 | 4.55 |
| U. S. (present Standard) | 8.00 | 3.75 | 2.25 | 67.50 | 1.000 | 4.60 |
| Oldenburg (Germany) | 8.65 | 4.15 | 1.95 | 70.45 | 1.045 | 4.80 |
| American (So-called Roman) | 11.80 | 3.95 | 1.55 | 73.20 | 1.085 | 5.00 |
| Marseilles (France) | 8.65 | 4.35 | 1.95 | 73.85 | 1.095 | 5.00 |
| Paris (France) | 8.65 | 4.15 | 2.35 | 84.55 | 1.250 | 5.75 |
| English | 8.65 | 4.30 | 2.55 | 95.10 | 1.410 | 6.45 |
| Paris (Baugtrard) | 8.65 | 4.35 | 2.55 | 96.00 | 1.420 | 6.55 |
| Swedish (Western) | 9.25 | 4.40 | 2.35 | 96.35 | 1.425 | 6.55 |
| Old Roman | 7.85 | 7.85 | 1.55 | 97.65 | 1.445 | 6.65 |
| English (Staffordshire) | 9.00 | 4.60 | 2.50 | 103.00 | 1.525 | 7.00 |
| Holland (Roermonder) | 9.45 | 4.65 | 2.35 | 103.70 | 1.535 | 7.05 |
| English | 9.00 | 4.35 | 2.70 | 104.50 | 1.550 | 7.10 |
| Paris (France) | 9.05 | 4.35 | 2.75 | 108.05 | 1.600 | 7.35 |
| Switzerland | 9.85 | 4.70 | 2.35 | 109.85 | 1.625 | 7.50 |
| Russia | 9.85 | 4.70 | 2.35 | 109.85 | 1.625 | 7.50 |
| German (Standard) | 9.85 | 4.70 | 2.55 | 119.00 | 1.765 | 8.10 |
| Alsace (France) | 9.85 | 4.70 | 2.55 | 119.00 | 1.765 | 8.10 |
| Spain | 9.85 | 4.70 | 2.55 | 119.00 | 1.765 | 8.10 |
| England (North) | 9.00 | 4.55 | 3.00 | 122.10 | 1.810 | 8.30 |
| Mexico | 10.25 | 5.10 | 2.55 | 134.05 | 1.985 | 9.10 |
| Upper Italian | 10.25 | 5.00 | 3.55 | 181.35 | 2.685 | 12.35 |
| Norway and North Sweden | 11.80 | 5.70 | 2.95 | 199.05 | 2.950 | 13.55 |
| Old German Cloister | 11.20 | 5.30 | 3.35 | 199.55 | 2.955 | 13.60 |
| Alsace (France) | 14.15 | 7.10 | 2.55 | 257.00 | 3.805 | 17.50 |
| Babylonian | 13.00 | 13.00 | 3.15 | 531.60 | 7.875 | 36.20 |
| Old Roman | 17.70 | 17.70 | 1.95 | 617.80 | 9.150 | 42.05 |
| Old Greek | 11.65 | 11.65 | 6.20 | 844.70 | 12.515 | 57.50 |
| Old Greek | 23.30 | 23.30 | 23.30 | 12650.00 | 187.550 | 862.00 |

From this table, it is observed that English and Continental building bricks are usually larger than those made in the United States.

There has been considerable thought devoted to the possibility of changing the standard size of building brick in the United States, so that each dimension plus a mortar joint will be a simple proportion of some module. Frederick Heath, Jr., an early proponent of the modular unit, has proposed a module of 8 inches for brick masonry. Modular planning simplifies the dimensioning of brickwork. Center-to-center of joint dimensions horizontally are multiples of 4 inches. Vertically, three courses space 8 inches. Cooperating manufacturers produce brick to fit these dimensions and it is anticipated that windows, door frames and other structural elements will be standardized in conformity.

The American Society for Testing Materials' Committee C-15 on Manufactured Masonry Units instructed (June 28, 1938) its Working Committee on Modular Sizes to prepare a list of masonry unit sizes based on a 4 inch module. The Working Committee suggests (Nov. 1, 1938) a brick of nominal modular dimensions 2 2/3 x 4 x 8 inches. Without the mortar joint, the average brick sizes would be grouped approximately as follows:

**MODULAR BRICK SIZES**

| Compensation for Mortar | Depth Inches | Width Inches | Length Inches |
|---|---|---|---|
| ½" Joint | 2 ⅛ | 3 ½ | 7 ½ |
| ⅜" Joint | 2 7/24 | 3 5/8 | 7 5/8 |
| ¼" Joint | 2 5/12 | 3 ¾ | 7 ¾ |

Subsequently, the American Standards Association has formed a Sectional Committee for its Project A62, Co-ordination of Dimensions of Building Materials and Equipment. This committee functions under the joint leadership of the American Institute of Architects and the Producers' Council and in cooperation with Modular Service Association. The latter is a non-profit corporation which makes available to the industry the many years of research by the late Albert Farwell Bemis. The Association endorses 4 inch modular dimensioning and brick sizes given above.

## 107. APPEARANCE, TEXTURE AND COLOR

The character of the clay, or mixtures of clays or shales, the manner of molding or forming, the manner and degree of burning in the kiln all have an influence on the appearance of the finished brick. It is unnecessary to trace these various influences. The finished brick from different parts of the country and often from various positions in some kilns have different colors and surface textures.

Natural clays and shales vary in composition with the geography of the country. Red-burning clays are the more prevalent, but buff burning and other clays are also widely used, partly on account of their color and partly because coloring materials can be added to them and so produce a wide variety of color effects. Different manufacturing processes produce different surface textures. And so the whole sweep of color, in smooth to rough textures, is available, from the pure severe tones of pearl grays or creams, through buff, golden and bronze tints to a descending scale of reds, down to purples, maroons, and gun-metal black.

The natural color of clay and shale bricks depends upon the chemical composition of the raw material and the degree of burning. Variation in the iron content of the clay is the usual cause of variation in color of red bricks, where burning is constant. McBurney used 182 of Ridgway's color names, to describe 3582 specimens of common

brick. However, 152 out of these colors were hues, tints and shades, more or less mixed with gray or the spectral range Red to Yellow. Ninety-nine per cent of the common brick production of the United States classifies as reds, buffs and creams. In the one per cent remaining, is found almost the complete range of the spectrum. For example, there naturally occurs in Wyoming bricks, a pear green color resulting from the occurrence of Vanadium in the clay.

## TABLE 2

## DESCRIPTION BY DISTRICTS

| Name of District | Number of Plants Samples | General Color Range |
|---|---|---|
| Maine, New Hampshire, Vermont | 9 | Deep Reds and browns with some fawn color and drab. |
| Massachusetts | 15 | As above. Also medium reds and orange reds. |
| Connecticut | 12 | Deep red and deep orange red to medium red and orange red with some browns, fawns and drabs. |
| Hudson Valley | 18 | Smoked medium red and medium orange red. Blotching of dull greens and pinks. |
| New Jersey | 10 | In part like Hudson Valley and medium red and orange red on light brown. |
| Eastern Pennsylvania and Delaware | 20 | Dark and medium red and purplish red. |
| Western Pennsylvania and West Virginia | 10 | Dark reds, usually dulled with gray. Some dark flashing. |
| Northern Ohio | 11 | Highly variable. |
| Detroit | 6 | In part resembles Connecticut, otherwise medium and light red orange. |
| Chicago | 9 | Light buff and cream with irregular pinkish markings. |
| Wisconsin | 6 | Pale buff to light cream. Some grades resemble Hudson Valley production. |
| Kentucky | 6 | Dark reds and purplish reds. |
| Illinois and Missouri | 12 | Medium and dark red and medium red orange. Some dark purplish red. |
| Nebraska | 5 | Medium and light red and red orange with some blue black slag. |
| Colorado | 8 | Medium and light red and orange red. Some cream and brown. |
| California | 4 | Medium red and medium red orange. |
| Texas | 15 | Light red and light red orange. |
| Louisiana and Mississippi | 7 | Spotting and speckling of black brown common. Fawns, drabs, browns, medium and dark red, medium red orange and medium purplish red. |
| Alabama | 4 | Dark, medium and light red with some medium red orange. Flashed of dark gray and purple gray. |
| North Carolina | 4 | Dark and medium red and red orange with overlay of brown and flashes of drab. |
| Virginia | 8 | Dark to light red and red orange. |
| Maryland and District of Columbia | 7 | Dark to medium red. |

Certain chemicals are frequently added to the clay or in the kiln to produce color effects. McBurney presents the following classification of colors defined in terms of Ridgway's scale:

"LIGHT RED covers the range Light Coral Red through Coral Red to Dragon's Blood Red. These are clear reds representing first order graying.

"MEDIUM RED corresponds to the several first and second grayings of reds and orange reds such as Pompeian Red, Hydrangea Red, Corinthian Red and Etruscan Red.

"DARK RED includes such colors as Brick Red, Mineral Red, Prussian Red, Deep Corinthian Red and Ocher Red. The brick production of certain districts shows red with noticeable tones of purple.

"MEDIUM PURPLISH RED corresponds to Livid Brown and Russet Vinaceous which are third grayings of red and orange red-orange.

"DARK PURPLISH RED covers Indian Lake, Dark Livid Brown and Deep Livid Brown.

"LIGHT RED ORANGE describes Salmon and Flesh color.

"MEDIUM RED ORANGE should correspond to Cameo Brown and Vinaceous Russet.

"The following names from Ridgway are believed to be well-known: IVORY, YELLOW, CREAM, BUFF, FAWN, PURPLE DRAB and QUAKER DRAB."

Table No. 2 was constructed to give a popular description of the common brick production of the United States.

The question of rating the quality of brick by color will be discussed under the section Testing of Brick.

The combination of color and texture may be of almost endless variety. Texture is the surface effect or appearance of a brick, apart from color. Unfortunately, it is practically impossible to give recognizable descriptions of characteristic textures, so this section will be limited to mentioning the effect of manufacturing processes on textures. As would be expected, sand-molded, soft-mud bricks possess a rough texture due to adhering sand. The edges of soft-mud bricks are seldom sharp, and the only really rough surface is the flat side where the excess clay was cut after filling the mold. Dry-press brick are characterized by very smooth surfaces and sharp edges and corners. Frequently, the individual grains of clay are still distinguishable, giving a finely mottled or granular surface, though quite smooth. Stiff-mud bricks, unless given a treatment designed to produce an artificial roughening are characterized by very smooth surfaces except on the end or flat, whichever is wire-cut. Stiff-mud brick are made with a wide variety of special textures, many of which are patented and the names copyrighted. The degree of burning also influences the texture. Certain types of over-fired or clinker bricks have a black glaze due to partial melting of the brick's surface.

## 108. SPECIFIC GRAVITY AND WEIGHT

The bulk specific gravity of building bricks apparently ranges between 1.57 and 2.32 according to available data. A standard brick may range in weight from less than 4 to almost 6 pounds; the usual weight being 4½ to 5 pounds. The determination of specific gravity is very important to the ceramist, but apparently its only value to the structural engineer or architect lies in enabling him to calculate the weight of masonry.

## 109. STRENGTH

Strength is the resistance to rupture under a gradually increasing load. For a number of reasons, different methods of measuring strength will sometimes result in different ratings for a given series of bricks.

Strength, in general, is influenced by the nature of the raw material, the degree of burning and to a slight extent the manner of forming the unit. Well-burned brick are

stronger than the under-burned or salmon brick made from the same clay or shale and by the same process of manufacture. However, this rule does not permit the grading of bricks into well-burned and under-burned on the basis of strength alone when more than one make of brick is considered, because the strength of brick, as previously stated, is also influenced by the nature of the raw material and the method of manufacture. To illustrate: A well-burned shale brick, reported in a paper by McBurney, gave a flat compressive strength of 10,170 pounds per square inch, while the salmon brick of the same manufacturer tested 6,093 pounds per square inch. Compare with this, the flat compressive strength of a well-burned surface-clay brick, which was 3,520 pounds per square inch. The strength of the under-burned bricks from the same surface clay averaged 1,860 pounds per square inch.

For weighted averages and dispersions of the compressive and transverse strength of the building brick produced in the United States, the reader is referred to McBurney's and Lovewell's paper in Proceedings A.S.T.M., Vol. 33, Part II.

The relationship existing between strength of brick and strength of masonry will be discussed in Chapter 3. The following sections are confined to the properties of the brick unit.

## 110. COMPRESSIVE STRENGTH

The compressive strength of clay and shale bricks tested on edge according to the above-mentioned paper by McBurney and Lovewell, ranges from a minimum of 1,142 pounds per square inch (a salmon brick) to 18,950 pounds per square inch. The weakest well-burned brick represented by tests of more than a single specimen gave an average of 3,032 pounds per square inch tested on edge.

Table 3, taken from McBurney's and Lovewell's paper, is based upon samples representing 37 per cent of the 1929 brick production (clay and shale) of the United States. As indicated, 92 per cent of the bricks have flat compressive strength of 3000 p.s.i. or better, excluding salmons. The weighted average of all brick both hard and salmon is 7246 p.s.i., "hard brick" only, 7434 p.s.i. and salmon, 4094 p.s.i. From the distribution data given, approximately 6 per cent of the production classify as 1250 to 2500 p.s.i., 20% as 2500 to 4500 p.s.i., and 74% as 4500 p.s.i. or over. Approximately 40 per cent of the production is 8000 p.s.i. or better in compressive strength.

## 111. TRANSVERSE STRENGTH (Modulus of Rupture)

The transverse strength of a brick is its resistance to a load when the brick acts as a beam supported at both ends. The value for transverse strength is usually expressed as the Modulus of Rupture. The Modulus of Rupture is calculated by means of the following formula.

$$\text{M.R.} = \frac{3wl}{2bd^2}$$

Where "M.R." is the modulus of rupture, "w", the total load in pounds at point of breaking, "*l*", the span in inches between supports of the brick (7 inches is the present standard span), "b" is the breadth of the brick (3¾ inches for a standard size brick), and "d" is the depth of the brick (2¼ inches for the standard size). When a standard size brick is tested, this formula reduces to M.R. = 0.553w.

Laminar structure in an end-cut, stiff-mud brick apparently raises the transverse strength as compared to its compressive strength. The lamination in a side-cut brick lowers the transverse strength in relation to its compressive strength. There is no general rule for converting values for compressive strength to transverse strength and vice versa. Tests by the Bureau of Standards on transverse strength, reported as modulus

of rupture, ranged from a minimum of 141 pounds per square inch (a salmon) to a maximum average of 2,890 pounds per square inch. The lowest average modulus of rupture reported for well-burned bricks, where more than one specimen was tested, was 515 pounds per sq. inch. Nearly seven per cent of the samples were reported as exceeding 2101 lb./sq. in. and only 0.37 per cent as less than 150 lb./sq. in. Improvements in methods of manufacture are constantly being made which increase all strength properties of brick, including the modulus of rupture.

Table No. 4, which is contained in Vol. II of A.S.T.M. Proceedings, Vol. 33, as part of a report by McBurney and Lovewell, gives the average values for properties of brick from several localities in the United States. It will be noted that these physical properties of brick vary with the locality.

TABLE 3

DISTRIBUTION OF STRENGTH PROPERTIES OF BRICK FROM ALL PARTS OF THE UNITED STATES

| Compressive Strength, Flatwise | | Modulus of Rupture | |
|---|---|---|---|
| Range, lb. per sq. in. | Percentage of production within range | Range, lb. per sq. in. | Percentage of production within range |
| 21001 to 22500 | 0.46 | 2101 to 3450 | 6.95 |
| 19501 to 21000 | 0.69 | 1951 to 2100 | 3.00 |
| 18001 to 19501 | 0.46 | 1801 to 1950 | 2.74 |
| 16501 to 18000 | 2.04 | 1651 to 1800 | 7.57 |
| 15001 to 16500 | 1.49 | 1501 to 1650 | 8.34 |
| 13501 to 15000 | 3.71 | 1351 to 1500 | 5.34 |
| 12001 to 13500 | 4.76 | 1201 to 1350 | 7.12 |
| 10501 to 12000 | 7.78 | 1051 to 1200 | 10.55 |
| 9001 to 10500 | 8.61 | 901 to 1050 | 10.44 |
| 7501 to 9000 | 11.92 | 751 to 900 | 13.60 |
| 6001 to 7500 | 15.47 | 601 to 750 | 11.74 |
| 4501 to 6000 | 16.81 | 451 to 600 | 7.52 |
| 3001 to 4500 | 17.97 | 301 to 450 | 4.35 |
| 1501 to 3000 | 7.46 | 151 to 300 | 0.37 |
| 0 to 1500 | 0.36 | 0 to 150 | 0.37 |
| Total per cent | 99.99 | | 100.00 |

The above tabulation is for a so-called hard brick, as classified by the manufacturer.

## 112. TENSILE STRENGTH

Very few tests on the strength of individual bricks in tension have been reported. McBurney made tests on six makes of brick and considers that there is a very fair relation between the tensile and transverse strengths of brick. His data show that the tensile strength is between 30 and 40 per cent of the transverse strength.

## 113. SHEARING STRENGTH

The Bureau of Standards made some tests by the so-called "punching shear" method on five makes of brick. The shearing strength ranged from a minimum of 1110 pounds per square inch to a maximum of 3550 p.s.i. The ratios of strength in shear to strength in compression flatwise ranged from 30 to 45%.

## 114. EFFECT OF WETTING ON STRENGTH OF BRICK

Twenty-three tests on brick reported in the Watertown Arsenal Test of Metals (1895), give an average reduction in compressive strength of fifteen per cent from the average of the compressive strength dry. Two of the individual makes of brick were stronger wet than dry. In 1929, tests were reported from the Bureau of Standards on the wet compressive strengths, flat and on edge, and the transverse strengths of four makes of brick in comparison with the dry strengths of other samples of the same makes. With the possible exception of one make of brick, wetting did not affect the strength of the brick tested. It is probable that some of the reduction in compressive strength resulting from testing wetted brick is explained by the effect of wetting on the capping material used. It would be expected that material reductions in strength caused by wetting would be found with under-burned brick.

## 115. HARDNESS AND RESISTANCE TO ABRASION

The terms "hard" and "soft" are used rather loosely when applied to building brick. Usually these terms are intended to be synonymous with well-burned and under-burned respectively; however, the more descriptive terms, well-burned and under-burned, are to be preferred.

The Brinell hardness of brick has been measured, and was reported by J. W. McBurney —"The Relation of Brinell Hardness and Transverse Strength to the Compressive Strength of Building Brick" Journal of the American Ceramic Society, Vol. 13, No. 11, Nov. 1930. There was a slightly better relation between Brinell hardness and flat compressive strength than there was between transverse strength and flat compressive strength.

The resistance to abrasion of bricks is affected by the degree of burning and by the nature of the raw material. The degree of resistance varies from extremely under-burned salmons at the low end, to the resistances of vitrified shale and fire clay brick, which are notably high. In general, it can be said that brick cover much the same range of resistance to abrasion as that obtained in natural building stones.

Abrasion and other physical property tests on building brick were made recently at the Bureau of Standards.

The abrasion tests were made using the portable abrasion machine developed by D. W. Kessler. The machine consists essentially of a hollow rotating shaft free to move vertically. The lower end of the shaft is fitted with a cylindrical foot, two inches in diameter, with a one-inch diameter hole through the center. Four radial slots, one quarter inch wide, are cut in the lower face of the shoe to allow the abrasive, supplied from the top through the hollow shaft, to flow continuously across the surface being abraded. In these tests the abrasive was 60-mesh carborundum. The total weight acting upon the foot was 4 kg. The shaft rotated at a speed of 100 R.P.M. and for a time of 5 minutes. The method of test consisted of supporting the brick on edge under the foot so that the foot rested upon the upper face with as level and normal a bearing as reasonably possible. The flow of abrasive was started and the machine operated until a smooth bearing surface was produced and the foot was in even contact with the surface of the brick. The brick was then cleaned by vigorous brushing and weighed to the nearest 0.01 gram. The brick was then replaced under the foot, taking care that the foot fitted into the previously abraded circle. The machine was then operated for 5 minutes, the brick. brushed clean and reweighed. The difference between the two weighings represents the loss caused by abrasion.

Results have been listed in Table 5 according to the abrasion loss in weight. It will be noted that there is a general relation between the weight loss by abrasion and other physical properties. In other words, the bricks with the lesser abrasive losses are

## TABLE 4

## WEIGHTED AVERAGES OF STRENGTH, WATER ABSORPTION, AND RATIO, OF 48 HR. COLD TO 5 HR. BOILING WATER ABSORPTION, FOR HARD AND SALMON BRICKS BY DISTRICTS AND CITIES

| District or City | Type of Brick | Average Compressive Strength, lb. per sq. in. | Average Modulus of Rupture, lb. per sq. in. | Average Water Absorption, Per Cent | | | Average Ratio, 48 Hour Cold to 5 Hour Boiling Water Absorption |
|---|---|---|---|---|---|---|---|
| | | | | 5 hour Cold | 48 hour Cold | 5 hour Boiling | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) |
| Maine, New Hampshire and Vermont | Hard | 10330 | 1505 | 5.7 | 6.3 | 8.7 | 0.70 |
| | Salmon | 5520 | 995 | 13.9 | 14.5 | 17.2 | 0.84 |
| Boston, Mass | Hard | 8240 | 1360 | 7.7 | 8.2 | 10.6 | 0.74 |
| | Salmon | 3710 | 645 | 16.4 | 17.1 | 19.6 | 0.88 |
| Massachusetts (except Boston) | Hard | 5940 | 1100 | 9.8 | 10.3 | 13.2 | 0.77 |
| | Salmon | 3410 | 700 | 15.7 | 16.4 | 19.8 | 0.84 |
| Connecticut | Hard | 7210 | 1155 | 12.9 | 13.5 | 16.2 | 0.82 |
| | Salmon | 4670 | 770 | 18.6 | 19.1 | 21.8 | 0.87 |
| Hudson Valley | Hard | 4740 | 745 | 14.0 | 14.6 | 17.7 | 0.80 |
| | Salmon | 2580 | 425 | 19.5 | 20.0 | 24.3 | 0.83 |
| New York (except Hudson Valley) | Hard | 4770 | 1005 | 14.4 | 15.4 | 18.3 | 0.83 |
| New Jersey | Hard | 5410 | 900 | 9.1 | 10.0 | 13.6 | 0.73 |
| | Salmon | 2080 | 530 | 17.7 | 18.9 | 22.1 | 0.86 |
| Delaware | Hard | 8040 | 1210 | 6.9 | 7.7 | 12.0 | 0.63 |
| | Salmon | 2800 | 450 | 12.9 | 13.8 | 16.0 | 0.86 |
| Baltimore, Md | Hard | 5990 | 1330 | 8.0 | 9.1 | 12.3 | 0.75 |
| | Salmon | 3130 | 645 | 17.3 | 18.2 | 20.9 | 0.88 |
| Maryland (except Baltimore, Md.) | Hard | 4440 | 960 | 8.4 | 9.2 | 13.4 | 0.68 |
| | Salmon | 2680 | 570 | 14.1 | 15.4 | 18.5 | 0.83 |
| Philadelphia, Pa | Hard | 6580 | 855 | 8.9 | 9.8 | 14.5 | 0.67 |
| | Salmon | 2520 | 355 | 16.7 | 17.9 | 22.9 | 0.79 |
| Eastern Pa. (except Philadelphia) | Hard | 8080 | 1415 | 7.8 | 8.7 | 11.8 | 0.73 |
| | Salmon | 3250 | 580 | 17.0 | 18.0 | 21.3 | 0.82 |
| Pittsburgh, Pa | Hard | 11060 | 2055 | 4.8 | 5.5 | 7.7 | 0.69 |
| | Salmon | 4880 | 955 | 11.4 | 12.1 | 14.1 | 0.85 |
| W. Va. and Western Pa. (except Pittsburgh) | Hard | 11450 | 1845 | 4.9 | 5.6 | 8.0 | 0.69 |
| | Salmon | 4810 | 940 | 11.7 | 12.5 | 14.6 | 0.85 |
| Cleveland, Ohio | Hard | 9820 | 1185 | 8.2 | 8.7 | 11.1 | 0.78 |
| | Salmon | 7780 | 1285 | 9.7 | 10.3 | 12.0 | 0.85 |
| Northern Ohio (except Cleveland) | Hard | 12110 | 1720 | 7.1 | 7.9 | 10.4 | 0.68 |
| | Salmon | 3910 | 620 | 12.6 | 13.5 | 14.6 | 0.79 |
| Detroit, Michigan | Hard | 4330 | 900 | 15.6 | 16.5 | 19.5 | 0.81 |
| | Salmon | 2540 | 555 | 18.9 | 19.4 | 20.8 | 0.93 |
| Wisconsin | Hard | 5330 | 1110 | 15.6 | 16.6 | 20.3 | 0.81 |
| | Salmon | 2320 | 470 | 24.4 | 25.2 | 28.0 | 0.90 |
| Chicago, Ill | Hard | 3450 | 1265 | 13.2 | 15.2 | 19.0 | 0.79 |
| | Salmon | 2400 | 420 | 20.0 | 22.4 | 26.5 | 0.85 |
| Illinois (except Chicago) | Hard | 13050 | 1580 | 5.2 | 6.2 | 8.0 | 0.75 |
| | Salmon | 6330 | 780 | 14.3 | 15.2 | 17.2 | 0.88 |

TABLE 4—Continued

## WEIGHTED AVERAGES OF STRENGTH, WATER ABSORPTION, AND RATIO, OF 48 HR. COLD TO 5 HR. BOILING WATER ABSORPTION, FOR HARD AND SALMON BRICKS BY DISTRICTS AND CITIES

| District or City | Type of Brick | Average Compressive Strength, lb. per sq. in. | Average Modulus of Rupture, lb. per sq. in. | Average Water Absorption, Per Cent | | | Average Ratio, 48 Hour Cold to 5 Hour Boiling Water Absorption |
|---|---|---|---|---|---|---|---|
| | | | | 5 hour Cold | 48 hour Cold | 5 hour Boiling | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) |
| Kentucky | Hard | 7780 | 1255 | 10.3 | 11.2 | 15.5 | 0.73 |
| | Salmon | 4010 | 720 | 14.7 | 15.8 | 19.0 | 0.84 |
| Southern Indiana | Hard | 16280 | 2360 | 1.8 | 2.3 | 5.4 | 0.37 |
| St. Louis, Mo | Hard | 11040 | 1465 | 6.0 | 6.7 | 9.1 | 0.70 |
| | Salmon | 5940 | 585 | 13.8 | 14.7 | 16.3 | 0.90 |
| Missouri (except St. Louis) | Hard | 14660 | 2020 | 2.9 | 3.8 | 5.7 | 0.61 |
| | Salmon | 6430 | 1170 | 12.9 | 13.8 | 15.8 | 0.88 |
| Nebraska | Hard | 8300 | 1025 | 8.7 | 9.7 | 13.3 | 0.73 |
| | Salmon | 2480 | 460 | 14.0 | 15.3 | 20.6 | 0.76 |
| Kansas | Hard | 15850 | 1915 | 3.2 | 3.8 | 5.1 | 0.59 |
| | Salmon | 4500 | 600 | 15.2 | 16.0 | 17.6 | 0.91 |
| Denver, Colo | Hard | 5050 | 620 | 10.4 | 11.3 | 13.5 | 0.82 |
| | Salmon | 4210 | 415 | 13.6 | 14.4 | 16.4 | 0.88 |
| Colorado (except Denver) | Hard | 7340 | 1145 | 10.3 | 11.1 | 13.3 | 0.83 |
| | Salmon | 4900 | 740 | 15.5 | 16.0 | 17.8 | 0.90 |
| Utah | Hard | 7490 | 1305 | 13.0 | 14.2 | 16.6 | 0.78 |
| Wyoming | Hard | 6310 | 1410 | 7.2 | 8.1 | 12.1 | 0.68 |
| | Salmon | 5680 | 300 | 13.4 | 13.9 | 16.3 | 0.85 |
| Washington and Oregon | Hard | 7600 | 1245 | 9.5 | 10.3 | 14.1 | 0.70 |
| | Salmon | 3240 | 580 | 15.9 | 17.0 | 20.1 | 0.85 |
| Los Angeles, Calif | Hard | 3690 | 600 | 11.2 | 11.8 | 14.2 | 0.83 |
| California (except Los Angeles) | Hard | 4640 | 735 | 11.9 | 12.7 | 15.2 | 0.84 |
| Texas | Hard | 7285 | 1020 | 11.1 | 12.0 | 14.3 | 0.82 |
| Mississippi, Louisiana and Arkansas | Hard | 6320 | 911 | 11.4 | 12.3 | 16.5 | 0.79 |
| | Salmon | 2580 | 485 | 10.1 | 11.1 | 16.8 | 0.67 |
| Alabama, Tennessee and North Carolina | Hard | 9700 | 1525 | 7.0 | 7.9 | 10.6 | 0.72 |
| | Salmon | 4510 | 750 | 17.6 | 18.9 | 21.0 | 0.89 |
| Richmond, Virginia | Hard | 9350 | 1315 | 7.4 | 8.3 | 11.1 | 0.74 |
| | Salmon | 4120 | 650 | 15.0 | 15.9 | 18.0 | 0.88 |
| Virginia (except Richmond) | Hard | 9470 | 1815 | 6.7 | 7.5 | 10.0 | 0.75 |
| | Salmon | 6560 | 1440 | 11.2 | 11.9 | 13.8 | 0.78 |
| Washington, D. C | Hard | 5700 | 875 | 8.3 | 9.4 | 12.9 | 0.72 |
| | Salmon | 3500 | 385 | 14.3 | 15.4 | 16.7 | 0.91 |
| Weight average of all samples, both hard and salmon | ..... | 7246 | 1154 | 10.12 | 10.92 | 13.99 | 76.4 |
| Weight average of all hard | ..... | 7434 | 1183 | 9.77 | 10.56 | 13.66 | 75.8 |
| Weight average of all salmon | ..... | 4094 | 678– | 15.94 | 16.77 | 19.41 | 85.1 |

usually those which are high in compressive strength and modulus of rupture and low in absorption. There are, however, several exceptions in each classification.

Probably one of the most severe exposures to abrasion in actual service is in brick storm sewers and conduits carrying water, containing a large percentage of sand or other abrasive materials at high velocities. Thousands of miles of such sewers and conduits have been built. Many of them have been in service for more than a century. From actual investigations, it has been found that well-burned brick, frequently with relatively high absorptions, continue to render satisfactory service after periods of more than fifty years.

TABLE 5

PHYSICAL PROPERTIES OF VARIOUS BUILDING BRICK

| Abrasive Rating | Type of Brick * | Abrasive loss in grams | Compressive Strength (flatwise) lb./sq. in. | Modulus of Rupture lb./sq in. | Water 24-hr. Cold | Absorption 5-hr. Boiling | C/B Ratio |
|---|---|---|---|---|---|---|---|
| 1 | SC, FC..... | 0.14 | 14,730 | 2335 | 0.75 | 2.2 | 0.34 |
| 2 | SM, C, WS, hard.. | 0.42 | 13,500 | 1170 | 3.8 | 5.7 | 0.67 |
| 3 | SC, S, Hard..... | 0.91 | 11,750 | 2015 | 3.2 | 5.5 | 0.58 |
| 4 | SM, C, SS, Hard.. | 0.98 | 11,600 | 1364 | 4.6 | 7.3 | 0.63 |
| 5 | SC, S, De-aired... | 1.08 | 16,330 | 820 | 3.2 | 8.8 | 0.36 |
| 6 | SC, C, Salmon.... | 1.26 | 9,230 | 1000 | 9.8 | 11.5 | 0.85 |
| 7 | SC, S, Lt. Hard... | 1.68 | 5,670 | 1495 | 10.2 | 11.8 | 0.86 |
| 8 | EC, C..... | 1.97 | 3,650 | 1660 | 12.1 | 15.9 | 0.76 |
| 9 | DP, S..... | 2.19 | 7,370 | 775 | 12.8 | 14.3 | 0.89 |
| 10 | SC, S, De-aired... | 2.34 | 6,540 | 540 | 9.4 | 11.0 | 0.85 |
| 11 | SM, C..... | 2.48 | 5,670 | 1085 | 11.6 | 16.3 | 0.71 |
| 12 | SM,C,WS,Lt. Hard | 3.17 | 9,200 | 1365 | 13.2 | 15.6 | 0.84 |
| 13 | EC, C..... | 4.49 | 5,590 | 770 | 12.7 | 17.8 | 0.71 |
| 14 | SC, C, Hard..... | 5.72 | 4,190 | 335 | 9.6 | 12.8 | 0.75 |
| 15 | SC, C..... | 5.92 | 4,245 | 1160 | 23.0 | 26.0 | 0.88 |
| 16 | SM, C..... | 5.94 | 4,130 | 625 | 19.9 | 21.9 | 0.91 |
| 17 | DP, C..... | 6.39 | 3,230 | 775 | 22.7 | 31.0 | 0.72 |
| 18 | SM, C, SS, Lt. Hard | 6.40 | 5,000 | 600 | 15.8 | 18.2 | 0.87 |
| 19 | SM, C, Hard..... | 7.54 | 4,350 | 595 | 17.7 | 21.2 | 0.83 |
| 20 | DP, C..... | 9.22 | 3,920 | 405 | 12.7 | 15.1 | 0.84 |
| 21 | SM, C, Salmon.... | 18.85 | 1,580 | 245 | 22.1 | 24.0 | 0.92 |
| 22 | SM, C, Salmon.... | 27.21 | 1,440 | 195 | 16.3 | 24.0 | 0.68 |

* DP—Dry Press; EC—End Cut; SC—Side Cut; SM—Soft Mud; C—Clay; FC—Fire Clay; S—Shale; SS—Sand Struck; WS—Water Struck.

## 116. MODULUS OF ELASTICITY

The results of the Watertown Arsenal tests show moduli of elasticity for building brick (not salmons) ranging from 1,400,000 to 5,000,000. Well-burned bricks do not show the permanent set or flow characteristics of cement products at loads not exceeding one-third of the ultimate. The ratio of lateral expansion to longitudinal compression (Poisson's Ratio) ranged from 0.0434 to 0.114.

## 117. THERMAL CONDUCTIVITY OF BRICKS

Table 6 presents results on the thermal conductivity of building bricks, the values being given in terms of coefficient of conduction "C", which is B.T.U. per hour per square foot of area for one inch thickness and for one degree's difference in temperature (Fahrenheit). These data are taken from the 1922 report of the Insulation Committee of the American Society of Refrigerating Engineers.

TABLE 6

THERMAL CONDUCTIVITY OF BRICK

| Description of Brick | Temp. of Sample (°F) | Coefficient of Conduction, "C" | Authority |
|---|---|---|---|
| Very Porous, Dry | 32 | 1.29 | Hencky |
| Very Porous, Dry | 68 | 1.371 | Hencky |
| Very Porous, 1.2 vol. % water | 68 | 1.17 | Cammerer |
| Very Porous, 5.8 vol. % water | 68 | 1.695 | Cammerer |
| Very Porous, 21.5 vol. % water | 68 | 2.743 | Cammerer |
| Hand made, dry | 32 | 2.662 | Poensgen |
| Hand made, dry | 60 | 2.743 | Poensgen |
| Machine made brick, dry | 123.4 | 3.34 | Knoblauch |
| Machine made, 0.8 vol. % water | 122.7 | 3.46 | Knoblauch |
| Machine made, 1.81 vol. % water | 110 | 6.64 | Knoblauch |
| Machine made, dry | 32 | 3.55 | Poensgen |
| Machine made, dry | 104 | 3.71 | Poensgen |
| Machine made, dry | 176 | 3.79 | Poensgen |

It should be borne in mind that the above values are for individual brick as units and not as brick masonry. Different methods and techniques of measurement are offered in explanation of their divergence. The average coefficient of conductivity ranges from 1.33 (Hencky) to 4.45 (Knoblauch). The average for all tests is 2.80. However, the following conclusions seem warranted:

1st. Wetting increases the thermal conductivity of brick.
2nd. Porous brick, dry, are better insulators than dense brick.
3rd. Increase in temperature increases the conductivity coefficient.

## 118. EXPANSION OF BRICKS DUE TO CHANGE IN TEMPERATURE OR MOISTURE CONTENT

L. A. Palmer gives in the following table, results based upon temperatures ranging from below freezing to room temperature, also the effect of total immersion for one month on linear expansion of brick. See also the Watertown Arsenal tests of metals, and fire resistance tests by Ingberg and Forster.

J. W. McBain investigated the behavior of burned clay products in comparison with certain other building materials from the standpoint of their tendency to take up or lose water with change in atmospheric humidity. He concludes that by comparison with other building materials, well-burned clay products are practically free from such tendency.

R. E. Stradling reports that the linear expansion of well-burned clay brick, previously dried, and then immersed in water is less than 0.01 per cent.

TABLE 7

EXPANSION OF BRICK DUE TO CHANGE IN TEMPERATURE AND MOISTURE CONTENT

| Brick No. | Description | Absorption Range | Degree of Burning | Linear Thermal | Expansion Wetting (%) |
|---|---|---|---|---|---|
| 1 | DP, S and C..... | 11.5—17.5 | Hard | .0000039 | 0.0027 |
| | | | Soft | .0000047 | 0.0125 |
| 2 | SMSC, S..... | 5.2—10.7 | Hard | .0000052 | 0.0005 |
| | | | Soft | .0000052 | 0.0005 |
| 3 | SMSC, S..... | 6.0—11.5 | Hard | .0000041 | 0.0005 |
| | | | Soft | .0000049 | 0.005 |
| 4 | DP, C..... | 9.5—16.0 | Hard | .0000047 | 0.0005 |
| | | | Soft | .0000050 | 0.003 |
| 5 | DP, FC..... | 4.2— 9.0 | Hard | .0000037 | 0.0005 |
| | | | Soft | .0000038 | 0.0005 |
| 6 | SMSC, FC..... | 4.5— 9.8 | Hard | .0000036 | 0.003 |
| | | | Soft | .0000038 | 0.010 |
| 7 | DP, C..... | 18.4—25.1 | Hard | .0000068 | 0.020 |
| | | | Soft | .0000085 | 0.025 |
| 8 | SM, C..... | 19.4—25.6 | Hard | .0000071 | 0.006 |
| | | | Soft | .0000072 | 0.015 |

DP (Dry press), SMSC (Stiff mud side cut), SM (Soft mud), S (Shale), C (Clay), FC (Fire clay).

## 119. POROSITY

Porosity and water absorption are not interchangeable terms. For one thing, porosity is a volume relation and water absorption is a weight relation. Water absorption, corrected for the specific gravity of the brick, is a measure of the apparent porosity, which may be defined as the ratio of the sum of volumes of the pores (or small cells in the brick which can be filled with water) to the total volume of the brick. The true porosity is the ratio of the sums of all pores or cells to the gross volume of the unit. This is determined on finely powdered material. The numerical value for apparent porosity tells little about the absorption properties of the brick units. These properties are to a considerable extent determined by the size, shape and number of the pores. The apparent porosity, when determined by boiling five hours, ranges from a minimum of a few per cent in a well-burned brick to a maximum of 45% in the extreme underburned salmon.

## 120. WATER ABSORPTION

The water absorption of a brick is defined as the weight of water, expressed as a percentage of the dry weight of the brick, which is taken up by the brick under a given method of treatment. Water absorption is determined by a number of methods, such as partially or totally immersing the brick in cold distilled water for various periods of time ranging from a few minutes up to several days; by boiling from one to five hours; or by treating the brick, while immersed, to alternate applications of vacuum and pressure. All these methods fill the pores of the brick more or less completely with water. The boiling and vacuum treatments obviously give a more complete pore filling and

hence a higher water absorption than the other methods. Table 8, taken from Vol. 33, Part II of the Proc. Am. Soc. Testing Mat., indicates that 3.87 per cent of all bricks tested had water absorptions less than 2.0 per cent when subjected to 5-hour cold total immersion. By the same test, only 0.11 per cent exceeded 26.0 per cent absorption, and none in excess of 28 per cent absorption. When water absorption was determined by the 5-hour boil method, 0.53 per cent of all specimens were under 2.0 per cent absorption and only 0.75 per cent fell within the limits 28.0 to 34.0 per cent. The individual minimum was 0.3 per cent.

## TABLE 8

## DISTRIBUTION OF WATER ABSORPTION AND C/B RATIO FOR BRICK FROM ALL PARTS OF THE UNITED STATES

| WATER ABSORPTION | | | | Ratio, 48 hr. Cold to 5 hr. Boiling Water Absorption | |
|---|---|---|---|---|---|
| Range, Per Cent | PERCENTAGE OF PRODUCTION WITHIN RANGE | | | Range | Percentage of Production Within Range |
| | 5 hour Cold | 48 hour Cold | 5 hour Boiling | | |
| 0 to 2.00 | 3.87 | 2.65 | 0.53 | 0.16 to 0.30 | 0.37 |
| 2.01 to 4.00 | 8.05 | 4.96 | 2.77 | 0.31 to 0.35 | 0.45 |
| 4.01 to 6.00 | 14.99 | 13.00 | 3.27 | 0.36 to 0.40 | 0.21 |
| 6.01 to 8.00 | 13.73 | 14.22 | 9.94 | 0.41 to 0.45 | 0.64 |
| 8.01 to 10.00 | 16.52 | 15.82 | 13.77 | 0.46 to 0.50 | 1.28 |
| 10.01 to 12.00 | 11.80 | 11.77 | 11.19 | 0.51 to 0.55 | 1.17 |
| 12.01 to 14.00 | 11.14 | 15.29 | 13.88 | 0.56 to 0.60 | 3.30 |
| 14.01 to 16.00 | 8.84 | 8.24 | 13.09 | 0.61 to 0.65 | 8.64 |
| 16.01 to 18.00 | 5.01 | 5.52 | 11.02 | 0.66 to 0.70 | 12.37 |
| 18.01 to 20.00 | 2.42 | 4.77 | 9.14 | 0.71 to 0.75 | 16.80 |
| 20.01 to 22.00 | 2.22 | 2.31 | 5.72 | 0.76 to 0.80 | 19.41 |
| 22.01 to 24.00 | 1.31 | 0.64 | 2.14 | 0.81 to 0.85 | 15.65 |
| 24.01 to 26.00 | 0.00 | 0.68 | 1.95 | 0.86 to 0.90 | 13.89 |
| 26.01 to 28.00 | 0.11 | 0.00 | 0.75 | 0.91 to 0.95 | 5.32 |
| 28.01 to 34.00 | 0.00 | 0.11 | 0.75 | 0.96 to 1.00 | 0.51 |
| Total per cent..... | 100.01 | 99.98 | 99.91 | | 100.01 |

The above tabulation is for a so-called hard brick, as classified by the manufacturer.

While in general, an under-burned brick will have a water absorption which is higher than the well-burned brick made from the same clay or shale and by the same method of manufacture, there is apparently little justification for the belief that water absorption alone is a direct measure of resistance to weathering.

Reference is made to the 1928 report of A.S.T.M. Committee C-3 and to paper by Krueger and McBurney for data which apparently justify the statement that water absorption of building brick, by itself, is not a measure of its durability or tendency to transmit water.

## 121. CAPILLARITY AND RATE OF ABSORPTION

The pores or small openings present in burned clay products function as capillaries tending to draw water into the body. This action in a brick has been referred to as its rate of absorption or suction, and recently the term, "penetrability", has been used to denote this action of capillarity. McBurney suggests two measures of penetrability; the volume-time, which is the amount of water absorbed per unit of time for partial immersion, and the height-time, which is the distance wetting takes place against gravity per unit of time. He points out that these two measures are not necessarily equivalent. Capillarity has little bearing on water transmission through brick masonry, but undoubtedly it does affect the adhesion of brick and mortar.

L. A. Palmer and D. A. Parsons, in their "Study of the Properties of Mortars and Bricks and Their Relation to Bond", report the rate of absorption tests on several makes of brick by partial immersion for short periods of time. Their data indicated that brick having an absorption of 20 grams in one minute, when the flat of the brick (30 sq. in.) was in contact with $\frac{1}{8}$" of water, developed maximum bond strength at the mortar joint. If brick have a rate of absorption greater than the 20 grams per min., they should be wetted sufficiently to approximate that ratio when laid.

## 122. PERMEABILITY

Permeability may be defined as the readiness with which a substance permits a fluid to flow through it and is measured by the rate at which a standard fluid such as water, air, or other gas, flows through a mass of unit area and unit thickness. As pressure is necessary to cause the flow, it is necessary to specify the pressure or head of the fluid. Permeability is not necessarily proportioned to porosity, because the shape and distribution of the pores control permeability more than the actual amount of the pore volume. Authorities agree that the permeability of sound bricks is a very minor factor in water transmission through masonry. Capillary suction is a force enormously greater than the pressures produced by wind velocity. Experiments have shown that the permeability of certain other types of masonry units to air was 380 times the permeability of brick units. For discussion of permeability, the reader is referred to reports by Raisch, Wilson, Searle, Parsons and Ketcham, Westman and Hursh.

## 123. RESISTANCE TO WEATHERING

Weathering agents that affect all building materials to some extent may be classified under the following heads: Temperature change in the absence of moisture; temperature change in the presence of moisture; cyclic wetting and drying; percolation of water; soluble salts; the atmosphere as a disintegrating agent and biological action. Brick and other structural clay products differ from other materials due to the fact that all of these are of minor importance, except possibly temperature change in the presence of moisture and soluble salts. The occasions are rare, but when there does happen to be any disintegration due to the action of soluble salts, the theory is that the pores contain salts in solution, which crystallize upon lowering of temperature, or evaporation. The crystals may enlarge and exert pressure on the walls of the pores as in the case of ice crystals.

The weathering action that has the most serious effect on practically all building materials is alternate freezing and thawing in the presence of moisture. Authorities agree that strength or water absorption are not accurate guides to the weatherability of a brick. Experiments at the National Bureau of Standards indicate, however, that any brick having an average compressive strength of more than 8000 p.s.i., or 5-hour

boiling absorption of less than 8%, or 24-hour cold absorption of less than 6.4% will successfully withstand 75 cycles of freezing and thawing. There are, of course, many makes of brick that resist the 75 cycles of freezing and thawing, which have compressive strengths less than 8000 p.s.i. and water absorption greater than the percentages mentioned. Any brick that resists the 75 cycle test will probably qualify for the most severe exposure conditions in the United States.

A physical property of a brick has been advanced, which has an important part as a measure of its resistance to weathering. It is the ratio of its apparent porosity to true porosity. The apparent porosity is determined by immersion in cold water for 24 to 96 hours, while the true porosity is determined from the specific gravity of finely powdered brick. A simplification has been proposed, using for apparent porosity, the absorption by 24-hour cold immersion, and substituting for true porosity, the 5-hour boiling absorption. This is known as the *saturation coefficient* (formerly called the C/B ratio) and is expressed as a decimal.

The saturation coefficient, in theory, is a measure of that proportion of the total available pore space which is filled when a brick is normally saturated with water. In other words, when the saturation coefficient is subtracted from unity, the remainder is an indication of the residual, unfilled pore space. Inasmuch as water expands some ten percent in freezing, there must be sufficient unfilled pore space available to allow for this expansion. Therefore, the saturation coefficient should not be greater than 0.90 in a brick intended for exposure to frequent freezing in the presence of moisture.

Results of the above mentioned Bureau of Standards experiments were reported by McBurney in the Proceedings of A.S.T.M., Vol. 35, Part I. His conclusions were that: (1) Neither strength, water absorption nor saturation coefficient, considered separately, provide a practical means of predicting the resistance of a brick to cycles of freezing and thawing; (2) Strength or water absorption in combination with the saturation coefficient provide a better method of rating this resistance, but the combination of all three gives the most accurate means found to date, for predicting the freezing-and-thawing resistance of a brick; (3) To withstand 75 cycles of freezing and thawing, a brick having a compressive strength of not less than 2500 p.s.i. and a water absorption by 5-hour boiling of not more than 20%, should have a saturation coefficient of not more than 0.80.

The tentative A.S.T.M. Specification for Building Brick (C62-37T) uses all three properties (compressive strength, water absorption and saturation coefficient) in grading, with the provision that any brick having a compressive strength greater than 8000 p.s.i., or a 5-hour boiling absorption less than 8% be placed in the top grade to withstand severe weathering, regardless of its saturation coefficient. The properties of brick from different localities vary considerably. Some brick that may not rate the highest grading for resistance to severe weather, according to the tentative specification, in actual construction have successfully withstood severe weathering for years. This fact was recognized by A.S.T.M. Committee C-15 in 1939 when Tentative Specification for Building Brick C62-37T was amended to provide that if brick which do not conform to the strength, absorption, and saturation coefficient requirements of the specification will successfully pass 50 cycles of freezing and thawing, they are to be judged upon the basis of this test rather than other requirements of the specifications. The following is the amendment to C62-37T as adopted by Committee C-15: "The requirements specified in 2 (a) for water absorption (5-hr. boil) and saturation coefficient (C/B) shall be waived provided a sample of five brick, meeting all other requirements, complies with the following requirements when subjected to fifty cycles of the freezing and thawing tests.—

| Grade | Results of 50 Cycles |
|---|---|
| SW | No breakage and not greater than 1 per cent loss in dry weight of any individual brick. |
| MW | No breakage and not greater than 3 per cent loss in dry weight of any individual brick. |

A particular lot or shipment shall be given the same grading as a previously tested lot, without repeating the freezing and thawing test, provided the brick are made by the same manufacturer from similar raw materials and by the same method of forming; and provided also that a sample of five brick selected from the particular lot has an average and individual minimum strength not less than a previously graded sample, and has an average and individual maximum water absorption and saturation coefficient not greater than those of the previously tested sample graded according to the freezing and thawing test."

The Sodium sulphate test is sometimes used as a test for resistance to weathering, but apparently such a test is rarely used today for structural clay products. Actual freezing and thawing test requirements are occasionally specified, but there is no standard practice either with respect to the method of conducting the test, the number of cycles required, or the criteria of failure. Actual freezing and thawing test requirements are impractical for commercial specifications.

Practically all well-burned brick will successfully resist disintegration due to exposure to severe weather. The architect, engineer or builder is cautioned, however, to use only the brick recommended by the manufacturer for severe exposure. Underburned brick may be used as back-up or for interior walls and partitions.

## 124. EFFLORESCENCE

Inasmuch as the raw material is a clay, shale or fire clay of reasonable purity, and the brick is burned to maturity, *a high degree of chemical inertness characterizes brick and other burned clay bodies.* If, however, in rare instances, the clay or shale contains a considerable quantity of limestone or soluble salts, certain phenomena may take place. Lime nodules in under-burned brick may undergo expansion when wet, causing the brick to split or spall. Thoroughly wetting the brick is effective in disclosing any with such defects.

Soluble salts may be the source of efflorescence. It should be emphasized, however, that in the majority of cases the original source of such salts is not the brick unit, but the mortar materials, plaster or certain other building units or materials used with the brick.

If a brick contains sufficient soluble salts to produce efflorescence from these salts, it will usually give a positive indication by the wick test, which is described as follows:

The following test for efflorescence, known as the "Wick Test" and described in National Bureau of Standards, Research Paper 1015, may be used to determine the source of soluble salts in brick or mortar ingredients. Since most cements contain some soluble salts, it may not be practical to specify that all mortar materials shall be free from such salts. The test is of value, in determining the relative soluble salt content of the various masonry materials.

The following is quoted from Research Paper 1015:

> "At the National Bureau of Standards, the wick test consists in standing a whole brick on end in 0.5 inch of distilled water, maintained at approximately a constant level by inverting a flask of

water over the tray, with the mouth of the flask held at the desired level. The brick, except that portion in contact with water, is exposed to the air of the laboratory at ordinary temperature. At the end of 5 days the brick is dried in an oven at 105° to 110°C for at least 18 hr., and the amount of efflorescence is estimated by visual comparison with an untreated specimen.

The amount of efflorescence is rated on a scale consisting of six classifications or grades, described as follows:

| | | |
|---|---|---|
| None | (0) | No observable difference in the appearances of a brick after test and before. |
| Trace | (1) | Efflorescence barely distinguishable by careful comparison. |
| Slight | (2) | Observable. Not sufficient efflorescence to materially affect the appearance when viewed at a distance of approximately 6 feet. |
| Moderate | (3) | Distinct coating, but the original color of the brick distinguishable under the efflorescence. |
| Considerable | (4) | The original color the brick masked by the efflorescence. |
| Abundant | (5) | Efflorescence in such quantity that it may be brushed off readily." |

Regarding these ratings, the authors state:

> "It is considered that bricks that are not liable to effloresce correspond to the None, Trace and Slight of this grading. Bricks that can only be recommended with reserve from point of view of efflorescence, represent Moderate and possibly Considerable, and bricks, the use of which would constitute 'disaster', correspond to Abundant."

Another test for efflorescence is a chemical analysis of a brick. Palmer states that a brick containing 0.05 per cent or less of water-soluble sulphuric anhydrite in the outer ⅛" of its surface is fairly safe from efflorescence even under conditions favorable for its development. This test, however, is not as easily performed as the wick test.

In a portion of the survey reported by McBurney and Lovewell, 5-day wick tests were made for liability to efflorescence. Observations were made both wet and dry. The following summarizes the results. Of 580 grades classified by the manufacturers as hard and well-burned, 59% rated None, 14.5% Trace, 14.5% Slight, 9.5% Moderate, 2.5% Considerable, and 0.5% Abundant. The above results have not been weighted. Weighting would increase the percentage of production free from efflorescence, and decrease the percentage of Moderate to Abundant tests results. In general, well-burned bricks are less likely to develop efflorescence than under-burned bricks. When the rating of a brick is "Slight" or less by this test, some other source of efflorescence must be blamed when noticeable efflorescence appears on the masonry.

For the literature on efflorescence, reference is made to the bibliography prepared by Jackson and later papers on this subject by Wilson, McBurney and Parsons.

## 125. MISCELLANEOUS PROPERTIES

### (a) Sound Reduction

V. L. Chrisler reports that "there is a straight line relation between the logarithm of the weight of the material per square foot and the sound reduction factor measured

in decibels." It follows, therefore, that brick has excellent properties in reducing airborne sounds. Sound reduction is discussed at greater length in Chapter 3.

**(b) Electrical Conductivity**

R. C. Lewis reports the electrical conductivity of building brick as ranging from 0.5 to 11 microamperes at 25.1 KV, which is of the order of air.

## 126. TESTING OF BRICK

Compressive and transverse strengths, together with 24-hour cold immersion and 5-hour boiling water absorption tests have been standardized by the American Society for Testing Materials. See Standard Methods of Testing Brick, (C67-37).

The Ring Test is frequently used in the field to check the quality of a brick. This consists of striking one brick with a trowel, hammer or against another brick and noting the sound. Well-burned brick usually give a rather clear, resonant sound or metallic "clink", while definitely under-burned brick give a duller sound, resembling that of striking a block of wood. Like a number of other tests, however, the results are largely relative. There are exceptions to this test, for some well-burned brick do not "ring." The difference in raw materials and methods of manufacture may make sufficient variation in the results, so that the ring test cannot always be relied upon, other than for the grading of the output of a particular plant.

Where brick are made from a shale or clay that is constant in composition and nature, the color of the burned brick provides a very useful guide to the degree of burning and, therefore, to the water absorption, strength and durability of the bricks. Red burning (i.e. iron-bearing) clays almost universally have a depth of color proportional to the degree of burning. The darker colors are usually associated with the harder-burned brick, and the "salmon" color with the underburned brick. However, light-colored brick from fire clays or semi-fire clays may be very hard and well-burned. Hence, color alone does not suffice for a grading of different makes of brick, but is quite valuable for the purpose of grading a shipment or kiln of the product of one manufacturer.

## 127. SECOND-HAND BRICK

Brick are noted for their durability and it is only natural to find them being salvaged from old buildings and used again. Many architects and engineers, however, object to the use of second-hand brick for the following reasons: (1) Seldom uniform in quality because the under-burned brick, originally used as backup or in partitions, are generally mixed in with the well-burned exterior brick; (2) Not sufficiently clean and the old mortar and grime, adhering thereto, does not permit a good construction; (3) Frequently an unknown quantity with regards to knowledge of previous use and may have been subjected to chemicals or vapors which may cause the brick to discolor or effloresce.

If second-hand brick are used, the sorting of the salvaged brick should not be intrusted to a wrecking crew laborer. A trained man should direct the sorting and separation of the brick according to the original use and location in the structure. All brick should have the mortar chipped off and then brushed as clean as possible with wire brushes before using again.

It is very important that every precaution be taken to eliminate any possibility of under-burned or undesirable brick getting into the assortment intended for the facing of walls. Only a few soft brick in the face of a wall, when they disintegrate or start spots of efflorescence, will mar the appearance of the whole wall. More than a few of them may cause grave structural weaknesses.

# CHAPTER 2

# MORTAR

## 201. DEFINITION:

Mortar for Clay Products Masonry may be defined as a mixture of cementitious materials, with or without the addition of plasticizers or admixtures, and aggregate, reduced to a plastic state by the addition of water and used to bind masonry units together. So-called acid resistant joining compounds, the principal ingredients of which are asphalt, coal-tar derivatives, sodium silicate or sulphur, are not included in this discussion.

A mortar is termed grout when sufficient additional water has been added to give it a fluid consistency and to cause it to flow readily.

## 202. CEMENTITIOUS MATERIALS, PLASTICIZERS AND ADMIXTURES

Cementitious materials include portland cement, natural cement, slag cement, puzzolan cements, masonry cements, hydraulic lime, hydrated lime, lime putty and mixtures of these materials. Many cementitious materials also act as plasticizers and most plasticizers have some cementitious properties. Under plasticizers may be listed hydrated lime and lime putty, finely ground clay, and numerous proprietary compounds for use as integral waterproofers. Admixtures, in addition to plasticizers, may be integral waterproofers referred to above, most of which contain stearates or their equivalent, mortar coloring, and compounds intended to accelerate or retard the rate of hardening of the mortar.

## 203. PORTLAND CEMENT

Portland cement is defined in A.S.T.M. Specification C9-38 as the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous material, with no additions subsequent to calcination excepting water and untreated calcium sulfate. The A.S.T.M. Specification, referred to above, may be taken as a safe guide for specifying portland cement which will be satisfactory for use in mortar. This specification provides that the cement shall not develop initial set in less than 45 minutes when the Vicat needle is used, and that final set shall be obtained within 10 hours. The tensile strength requirements for standard mortar briquetts composed of one part cement and three parts standard Ottawa sand by weight are 275 lb. per sq. in. at the age of seven days (1 day storage in moist air, six days in water), and 350 lb. per sq. in. at the end of twenty-eight days (one day storage in moist air and twenty-seven days in water).

## 204. HIGH EARLY STRENGTH PORTLAND CEMENT

High-early-strength portland cement may be used in mortars where it is necessary or desirable that the masonry obtain a strength approaching its ultimate within a short time after it is laid. A.S.T.M. Specification C74-38 may be used as a guide in specifying high-early-strength portland cement. This specification makes the same requirements as to time of setting as the specification for portland cement, but contains strength requirements for standard mortar briquetts composed of one part cement and three parts standard Ottawa sand by weight of 275 lb. per sq. in. at the age of one day (one day storage in moist air) and 375 lb. per sq. in. at the end of three days (one day storage in moist air and two days storage in water).

## 205. NATURAL CEMENT

Natural cement as defined by A.S.T.M. Specification C10-37 is a finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas. Natural cements are obtainable in some localities at less cost than portland cement and due to the ingredients present in some natural cements which are not found in portland cements, they may produce mortars of greater plasticity than straight portland cement mortar. Strength of most natural cement mortars is less than the strength of portland cement mortars. The A.S.T.M. Specification for Natural Cement, referred to above, provides that the tensile strength of standard briquetts composed of one part cement and three parts standard Ottawa sand shall be 75 lb. per sq. in. at the age of seven days (one day storage in moist air and six days in water), and 150 lb. per sq. in. at the age of twenty-eight days (one day storage in moist air and twenty-seven days in water).

## 206. MASONRY CEMENTS

Masonry cements are largely proprietary mixtures some of which contain sand while others are to be mixed with sand on the job. Manufacturers seldom disclose the formulae for their cements. However, they are principally mixtures containing portland, natural, slag or puzzolan cement, and some plasticizing agent to which integral waterproofing has been added in some cases. Slag cement, puzzolan cement, and hydraulic lime are rarely used as separate ingredients for mortar in this country, but are usually found in masonry cements. The Federal Specification SS C-181b may be used as a guide in specifying masonry cements. Under materials and workmanship this specification permits the manufacturer "a wide range in the selection of materials and process of manufacture in order that cement of the prescribed quality may be produced."

## 207. LIME

Lime is both a cementitious and a plasticizing agent. Authorities differ as to the relative importance of these two properties. The cementing action of lime is due largely to the chemical action between the lime putty (calcium hydroxide or magnesium hydroxide) and the carbon dioxide in the air which produces calcium carbonate or magnesium carbonate. This carbonation of lime is a relatively slow process and consequently lime mortars may be expected to increase in strength over a long period of time. At one time straight lime mortars were used quite extensively; however, due to their slow rate of set and their relatively low strength, straight lime mortar has been largely replaced by modified portland cement or masonry cement mortars. An important function of lime, when used with portland cement in mortar, is as a plasticizer or water retaining agent. With a given cement and sand, the plasticity and water retentivity of the resulting mortar will vary directly with the plasticity of the lime.

## 208. QUICKLIME

As defined in A.S.T.M. Specification C5-26, quicklime for structural purposes contains a minimum of 95 percent of calcium and magnesium oxides which may be in varying proportions. Quicklime as such is never used for structural purposes, but must always be slaked to form a lime putty. The following method of preparing lime putty is taken from the Appendix of the A.S.T.M. Specification referred to above:

"Directions for Slaking — For quick-slaking lime, always add the lime to the water, not the water to the lime. Have enough water at first to cover all the lime completely. Have a plentiful supply of water available for immediate use — a hose throwing a good stream, if possible. Watch the lime constantly. At the slightest appearance of escaping steam, hoe thoroughly and quickly, and add enough water to stop the steaming. Do not be afraid of using too much water with this kind of lime.

For medium-slaking lime, add the water to the lime. Add enough water so that the lime is about half submerged. Hoe occasionally if steam starts to escape. Add a little water now and then if necessary to prevent the putty from becoming dry and crumbly. Be careful not to add any more water than required, and not too much at a time.

For slow-slaking lime, add enough water to the lime to moisten it thoroughly. Let it stand until the reaction has started. Cautiously add more water, a little at a time, taking care that the mass is not cooled by the fresh water. Do not hoe until the slaking is practically complete. If the weather is very cold, it is preferable to use hot water, but if this is not available, the mortar box may be covered in some way to keep the heat in."

While these methods, if followed, should give satisfactory results, various quicklimes act differently when slaked and the manufacturer's directions for slaking should be carefully followed.

## 209. HYDRATED LIME

Hydrated lime is produced from quicklime by adding sufficient water to convert the oxides to hydroxides. It is common practice in some localities to add the dry hydroxide to the mortar mix without any preliminary soaking of the lime. However, investigations indicate that the plasticity of dry hydrate putties is frequently improved by soaking for at least twelve hours. This procedure should be specified on all important work.

Research conducted at the National Bureau of Standards and recorded in their Research Paper No. 952, indicates that there is a wide variation in the plasticity and water retaining power of commercial hydrated limes; such a variation, in fact, that the investigators state "the flow after suction of mortars depends far more on the properties of the lime than on the cement-lime ratio."

A.S.T.M. Specification C6-31, Hydrated Lime for Structural Purposes describes a method of determining the plasticity of lime by means of the Emley Plasticimeter. The plasticimeter is essentially a machine which rotates a steel disc in contact with the lime paste on a porous base. The plasticity figure is a function of the time during which the disc rotates and the torque required to rotate it. The A.S.T.M. Specification, referred to above, contains no requirement for plasticity for masons hydrate. However, the plasticity requirement for finishing lime is that the lime shall have a plasticity figure of not less than 200.

A.S.T.M. Specification C6-31 may be used as a guide in specifying hydrated lime for structural purposes and if a highly plastic lime is desired, a plasticity requirement should be added to the specification.

## 210. LIME SOLIDS

Investigations by Professor Walter C. Voss ("The Characteristics of Lime"—National Lime Association) indicate that the weight of lime solids varies greatly in lime putties produced from different quicklimes. For the limes reported, this variation was from 31.3 lbs. to 63 lbs. of solids per cu. ft. of putty. Professor Voss found that the weight of the lime solids in a cu. ft. of putty was approximately constant for putties having the same weight and based on this assumption, he developed the following formula:

$$X = 1.7\ (W\text{-}62.0)$$

in which X = the weight of lime solids in a cu. ft. of putty and W = the weight of putty per cu. ft.

In this investigation, the average weight of lime solids was found to be 45 lbs. per cu. ft. and if it is desired to specify mortar on a lime solids basis, i.e., a mortar having a constant volume of lime solids, the volume of putty required may be obtained from the following equation:

$$V = \frac{26.5 \times L}{W - 62}$$

in which V = volume of putty required; L = volume of lime solids specified; W = weight of putty per cu. ft.

If a mortar were specified as one part portland cement, one part lime solids and 6 parts sand by volume, and it was found that the lime putty weighed 80 lbs. per cu. ft., the volume of lime putty required to obtain the 1:1:6 mixture, from the above equation would equal:

$$V = \frac{26.5 \times 1}{80 - 62} = 1.5 \text{ cu. ft. (Approximately)}$$

In above equations all wts. are in lbs., all volumes in cu. ft.

Quick limes which give the greatest yield in putties, that is which produce putties containing relatively low percentages of lime solids per cu. ft. of putty, as a rule are more plastic than putties containing higher percentages of lime solids. For this reason some designers specify lime putty on a straight volume basis rather than on the basis of lime solids. However, if it is desired to keep the actual proportion of lime in the mortar constant, the lime solid basis should be used.

## 211. FINELY GROUND CLAY

During recent years, finely ground clay or mortar mix, as it is sometimes called, has come into extensive use as a plasticizer for masonry mortars. In many localities, this material can be obtained at less cost than lime and the investigations which have been conducted to date indicate that very satisfactory results can be obtained through its use. M. G. Spangler reported tests on the strength and durability of mortar mix mortar in the Journal of the American Ceramic Society, Volume 16, No. 5, May 1933. These tests included tension, compression, freezing and thawing, bond strength, and absorption tests and the author concludes, "Mortar mix when used to replace up to 45 per cent or 50 per cent by volume of the cement, produces mortar at least as strong and as durable as mortars in which equal percentages of hydrated lime are used. The tests also served to emphasize the necessity of holding the amount of admixtures in mortars to the least percentage which will produce a workable mix." Investigations by H. G. Schurecht on the use of New York State clays in masonry mortars are reported in Bulletin No. 1, of the Ceramic Experiment Station, New York State College of Ceramics. Clays used in this investigation were dried at 230°F prior to grinding and were then pulverized in a grinder so that 80 per cent passed a 100 mesh sieve in the dry condition. The author concludes from this research, "This investigation demonstrates the possibility of producing mortars of equal or better quality than many now commonly used, and at less cost by employing finely ground New York State clays or shales. A composition which gave good results with practically all of the clays was composed of 0.25 clay or shale, 0.75 portland cement, and three sand based upon the volume basis." No nationally accepted specifications have as yet been developed for finely ground clay or mortar mix for use in masonry mortars. However, in the absence of definite knowledge as to the performance of the product, a fineness specification, that at least 80 per cent of the clay should pass a 100 mesh sieve in the dry condition would be desirable.

## 212. INTEGRAL WATERPROOFING

Integral waterproofers usually depend upon their water repellency for their effectiveness. As previously stated, most of these compounds contain some form of stearate. While some investigators recommend their use, many others question their effectiveness.

Tyler S. Rogers, formerly Technical Editor of American Architect, (American Architect Apr. 1935) concludes: "To an impartial observer waterproof portland cement mortars appear to have little advantage over the straight 1:3 cement mortars except for their water repellent characteristic and greater plasticity under the trowel. They produce satisfactory results according to some investigators and are of little or no value according to others." F. O. Anderegg, Building Consultant states in "Watertight Brick Masonry", The Architectural Record, September, 1931: "The results obtained in this research and in previous investigations have demonstrated repeatedly the superior results that can be obtained with properly distributed stearate waterproofer. The advantages have been found to be:

1. Improved workability.
2. Better wetting of the brick surface.
3. Prevention of shrinkage openings between brick and mortar by controlling the rate at which moisture is absorbed from the mortar.
4. Increased weather resistance by lowering the absorption.
5. Improvement in the flexibility of a mortar.

The disadvantages of using a waterproofer include a small reduction in compressive strength, a slight increase in cost and some water-repellency on mixing. The first effect is negligible in comparison to the improvements in the other properties. "The increase in cost will range from $.20 to $.50 per thousand brick, but this would be compensated by a 1% increase in speed of erection, readily securable with the increased workability and by the greatly improved quality of the job. The water-repellency is overcome readily by wetting the sand before adding the cement."

## 213. ADMIXTURES

Opinion varies widely concerning the effect on mortars produced by adding materials for the purpose of accelerating or retarding the rate of set, or increasing the workability. Some admixtures may not impair the strength as indicated by the seven day test and often, as claimed by their manufacturers, may improve the normal seven day strength. It sometimes happens however that the strength of mortar containing these admixtures does not show the customary relative increase at twenty-eight days. It is suggested therefore, that where strength is an important consideration preliminary tests be made on the specimens of mortar containing admixtures at both seven days and twenty-eight days. The use of common salt for lowering the freezing point, and sugar for retarding set should be prohibited.

## 214. COLOR

Mortar may be colored by using colored sand, such as ground granite, marble, or other stone. When the desired shade can be thus obtained, these are preferable to artificial colors, for natural sands and stones usually have a permanent color and do not weaken the mortar. White joints may be obtained with white sand, ground limestone or marble, using white cement. The color of the sand, in the finished joint, will, of course, be somewhat modified by uncolored cementing material.

Great care should be exercised in selecting the proper artificial color. Mortar is strongly alkaline and coloring matter should, therefore, be chemically inert or the color may fade or run when in the wall. Mineral colors are preferable.

The practice of mixing cement, sand and color by volume in a mortar box with a hoe is not generally recommended, because uniform batches are difficult to produce. Wherever possible, it is better to weigh all the ingredients and to measure the water. In any case, the manufacturer's directions should be carefully followed.

## 215. AGGREGATE

A.S.T.M. Committee C-12 on Mortars for unit masonry has recently published a proposed specification for aggregate for masonry mortars (Bulletin A.S.T.M., March, 1939). As defined in this specification, aggregate for use in masonry mortar consists of fine granular material composed of hard, strong, durable, mineral particles which are free from injurious amounts of saline, alkaline, organic or other deleterious substances. In the report of the Sub-Committee which prepared the specification, it is pointed out that there is substantial agreement as to the amount and nature of the deleterious substances permissible in a suitable aggregate. There is agreement also on the methods of test to determine the presence of these substances in the aggregate. However, there is a wide difference of opinion as to the size grading and grading limits for a suitable aggregate.

The following Table A represents what might be called a conventional grading and with slight modification has been used quite extensively in specifications:

**TABLE A**

**Percentage By Weight Passing Each Sieve**

| SIEVE | MINIMUM | MAXIMUM |
|---|---|---|
| No. 8 | 100 | |
| No. 16 | 65 | 90 |
| No. 30 | 35 | 70 |
| No. 50 | 15 | 40 |
| No. 100 | 0 | 10 |

This table was originally proposed for inclusion in the A.S.T.M. Specification. However, the discussion brought out that many aggregates which have been used with notable success do not conform to the grading limits of Table A. This discussion is summarized as follows in the report of the Sub-Committee:

"Both the limits on grading in Table A and the discussions from members of the sub-committee recognize the desirability of having a larger proportion of fine particles in aggregates for mortars than for the mortar component of concrete. The chief reasons are:

(1) The ratio of cement to fine aggregate in concrete usually is greater than for masonry mortars, and the fine particles of the cementing material in concrete mixtures tend to supplement any deficiencies in the amount of fine materials in the aggregates.
(2) The maximum size of fine aggregate for concrete usually is larger than for mortar aggregates and thus a smaller percentage of the finer particles is required for equal workability.
(3) Strength is of prime consideration in concrete, whereas, for mortars it is secondary to workability.
(4) Concrete mixtures, especially those used in pavements and floors, are well compacted after placing, whereas, mortars receive but little compaction and, therefore, must have such properties as will permit satisfactory placement with little manipulation.

"Although the need for relatively large proportions of fine particles is recognized, the desirability of having the amount of them vary with the size of the largest particles apparently has been overlooked in specifications. The maximum size of aggregates for mortars should depend upon the use of the mortars and vary with the thickness of the joints in the masonry. For joints of a given thickness there is some advantage in

using aggregates of the largest maximum size compatible with satisfactory workmanship. That tends to result in an economy in the use of cement and in a minimum volume change of the hardened mortar. However, it is not economical to exclude aggregates of suitable grading when they are either unusually coarse or unusually fine, if they are the only ones available at a low cost.

"In view of the foregoing, it seems that if the limits on grading are so broad as to include suitable aggregates for most uses they will not exclude poorly graded materials. A single set of grading limits, therefore, cannot be satisfactory unless supplemented by additional restrictions on grading. Perhaps the simplest way to provide for the acceptance of aggregates of suitable grading and at the same time to specify limits on grading which will reject those of poor grading is to have more than one set of limits. Then the grading limits would be in a form familiar to purchasers and sellers and no computations would be required to determine whether or not a particular aggregate meets the requirements. For example, there could be four sets of limits, each of which would define a material of suitable grading, and each differing from all of the rest in the proportions required to lie between two successive sieves. This form of specification is illustrated in Table B, the values of the grading limits being, incidentally, for materials having substantially geometrically similar particle size distributions after additions of a cement in commonly used proportions":

**TABLE B**

**Percentages By Weight Passing Each Sieve**

| SIEVE | GRADING 1 | GRADING 2 | GRADING 3 | GRADING 4 |
|---|---|---|---|---|
| No. 4 | 100 | -------- | ------- | ------- |
| No. 8 | 90 to 100 | 100 | 100 | 100 |
| No. 16 | 50 to 70 | 60 to 80 | 70 to 90 | 80 to 100 |
| No. 30 | 30 to 50 | 35 to 55 | 45 to 65 | 55 to 75 |
| No. 50 | 15 to 30 | 20 to 35 | 25 to 40 | 30 to 45 |
| No. 100 | 5 to 15 | 10 to 20 | 15 to 25 | 20 to 30 |
| No. 200 | 0 to 5 | 0 to 10 | 0 to 10 | 5 to 15 |

The final specification as submitted by Committee C-12 contains no definite grading specification but provides "aggregate for use in masonry mortar shall be uniformly graded from fine to coarse within the limits specified by the engineer." As a guide to the "Engineer" in fixing these limits, the following note is included:

IMPORTANT NOTE—The committee has found that sands of such widely different gradings have given satisfactory results in masonry mortar, depending upon the type of construction and the characteristics of the cementing material, that it has not attempted, at this time, to outline specific recommendations for grading. To do so would result in, on the one hand, limits so broad as to be meaningless or, on the other hand, limits so narrow as to prohibit the use of many materials which have been found to be satisfactory. Accordingly, the committee restricts its recommendations to the following outline of certain fundamental principles:

(a) The aggregate should be well-graded and contain sufficient of the finer sizes to provide a suitable degree of workability. Unduly fine aggregates should be avoided since they require the use of excessive quantities of mixing water or excessive quantities of cementing materials. In general, the use of an aggregate finer than one which is well graded up to a No. 16 sieve should be avoided.

(b) The maximum size of the aggregate should not exceed that consistent with the type of construction. The following limits are suggested:

| TYPE OF MASONRY CONSTRUCTION | MAXIMUM SIZE OF AGGREGATE* |
|---|---|
| Joints ½ in. or thicker | No. 4 sieve† |
| Joints of average thickness (such as for brick masonry) | No. 8 sieve† |
| Thin joints for units having cut or ground edges | No. 16 sieve† |

* A smaller maximum size may be used.
† A permissible variation of at least 5 per cent of material retained on the sieves designated as the maximum size should be permitted.

(c) The aggregate should contain a substantial proportion of material finer than a No. 50 sieve, although excessive quantities should be avoided. The following limits are recommended:

| MAXIMUM SIZE | PERCENTAGE PASSING NO. 50 SIEVE |
|---|---|
| No. 4 Sieve | 10 to 30 |
| No. 8 Sieve | 15 to 35 |
| No. 16 Sieve | 20 to 40 |

(d) To fill the gap which usually exists between the coarser particles in the cementing material and the No. 100 sieve size, it is desirable that some proportion of the aggregate, generally (but with possible exceptions) not to exceed about 20 per cent, be finer than the No. 100 sieve. Most washed natural sands contain relatively little material finer than the No. 100 sieve, and it is sometimes desirable to supply such finer sizes by blending with a second, and finer, aggregate. Aggregates containing sufficient material finer than the No. 100 sieve should be examined carefully to determine whether the fine material is of suitable mineralogical composition.

(e) The grading characteristics of the material available in the locality in which the work is to be done should be taken into account by the engineer. In order to insure a reasonable degree of uniformity in the aggregate for a given project the requirements should be so specified that ranges in the percentages of material passing the various sieves do not exceed the following:

**Recommended Permissible *Range* in Percentages Passing Each Sieve**

| SIEVE SIZE | MAX. SIZE NO. 4 SIEVE | MAX. SIZE NO. 8 SIEVE | MAX. SIZE NO. 16 SIEVE |
|---|---|---|---|
| No. 3 | 0 | .. | .. |
| No. 4 | 5 | 0 | .. |
| No. 8 | 15 | 5 | 0 |
| No. 16 | 20 | 20 | 5 |
| No. 30 | 30 | 30 | 25 |
| No. 50 | 20 | 20 | 20 |
| No. 100 | 10 | 10 | 10 |
| No. 200 | 5 | 5 | 5 |

(f) The percentages in the preceding table must not be confused with percentages passing or retained on the various sieves. The values given are ranges. For example, the following specification for grading of a fine aggregate with a maximum size of No. 16 (not given as a recommendation, but for illustration only) conforms to the recommendations in the preceding tabulation:

| SIEVE SIZE | PERCENTAGES PASSING EACH SIEVE |
|---|---|
| No. 8 | 100 |
| No. 16 | 95 to 100 |
| No. 30 | 60 to 85 |
| No. 50 | 20 to 40 |
| No. 100 | 0 to 10 |
| No. 200 | 0 to 5 |

From the proposed tentative specification submitted by Committee C-12 and the report of the Sub-committee it appears that suitable aggregates in many gradings or combinations of sizes will give satisfactory service when used in masonry mortar. It appears also that unlike fine aggregate for concrete, a relatively high percentage of fines (passing 100 mesh sieve) is desirable in aggregate for masonry mortar. In view of these facts the designer should familiarize himself with the grading characteristics of the aggregate available locally and if this aggregate does not fall within the grading range recommended in the A.S.T.M. specification, specify a grading which can be obtained through a modification of the material available, either by the elimination of a percentage of certain sizes or through the addition of fines. Ground clay is a suitable material for this purpose.

Committee C-12's proposed specification for aggregate in masonry mortar contains the following comments regarding the amounts of deleterious substance and methods of test:

(a) The amount of deleterious substance shall not exceed the following:

| | MAXIMUM PERMISSIBLE PERCENTAGE BY WEIGHT |
|---|---|
| Coal and lignite | 0.25 |
| Clay lumps | 1 |
| Shale, alkali, coated grains, soft or flaky particles | 1 |
| Other deleterious substances | 0.75 |
| Total deleterious substances | 3 |

(b) Aggregate subjected to the colorimetric test for organic impurities and producing a color darker than the standard shall be rejected unless it shall pass the following mortar strength test:

Aggregate shall be of such quality when subjected to the mortar strength test that it shall have a compressive strength at ages of 7 and 28 days of not less than 95 per cent of that developed by a mortar of the same water-cement-ratio and consistency made with the same cement and graded Ottawa sand having a fineness modulus of $2.40 \pm 0.10$.

## 216. METHODS OF TESTING AGGREGATE

The properties of aggregates may be determined in accordance with the following methods of test of the American Society for Testing Materials:

(a) Sieve Analysis — Tentative Methods of Test for Sieve Analysis of Fine and Coarse Aggregates (A.S.T.M. Designation: C41).

(b) Washing Test — Standard Method of Test for Determination of Amount of Material Finer than No. 200 Sieve in Aggregates (A.S.T.M. Designation: C117).

(c) Coal and Lignite — Tentative method of Test for Coal and Lignite in Sand (A.S.T.M. Designation: C123).

(d) Clay Lumps — Tentative Method of Test for Clay Lumps in Aggregate (A.S.T.M. Designation: C142).

(e) Organic Impurities — Standard Method of Test for Organic Impurities in Sands for Concrete (A.S.T.M. Designation: C40).

(f) Mortar Strength — Standard Method of Test for Structural Strength of Fine Aggregate Using Constant Water-Cement-Ratio Mortar (A.S.T.M. Designation: C87).

## 217. IMPERMEABLE MASONRY

Probably the most important single property of exposed masonry is its ability to resist the penetration of excessive amounts of moisture.

Both masonry units and mortar are usually porous to some degree and when exposed to the weather will absorb moisture; however, the amount of water which enters the masonry by capillarity is comparatively negligible and, if reasonable precautions are taken in design, will not affect the serviceability of the structure.

All authorities agree that **damaging moisture penetration or leakage occurs through openings in the mortar joints.**

These openings may be due to:

(a) Poor workmanship

(b) Separation cracks

## 218. WORKMANSHIP

Workmanship is undoubtedly the most important single factor in masonry construction, from the standpoint of moisture penetration. Poor workmanship, as herein referred to, includes unfilled or partially filled mortar joints, defective installation of flashing or caulking around door and window frames or other structural items. As used in this sense, poor workmanship does not imply that the mason is deficient in his work, he may be following instructions, but it does indicate that the assemblage is defective from the standpoint of resistance to moisture penetration.

Except for the fact that certain combinations of masonry units and mortar make the complete filling of joints easier for the mason, workmanship is not related either to masonry units or to mortar and, consequently, no mortar is a guarantee against leakage due to poor workmanship.

## 219. SEPARATION CRACKS

Cracks that may frequently be observed between mortar and brick have been attributed by some to the shrinkage and volume change of the mortar. While hair line cracks, particularly in the mortar joint may be due to shrinkage, the larger and from the standpoint of moisture penetration, more important cracks between mortar and brick are certainly due to other causes.

The cracks, referred to by some writers as "separation cracks" may be in existence immediately after the brick are laid, or they may appear at a later date. Tooling the mortar joint will temporarily seal separation cracks, but as the layer of compacted mortar on the surface of the joint is eroded away, the cracks may appear. Authorities differ as to the cause of separation cracks, however they appear to be due to one or both of the following:

(a) Incomplete bond due to separation of water from mortar — if, after the mortar bed is spread, water separates from the mortar and rises to the top, the bond between the mortar and the next course of brick will be "patchy," due to the entrapped water.

When this water evaporates, there will be portions of the brick not in contact with the mortar and subsequent freezing and thawing in the presence of moisture may destroy the bond entirely.

(b) Incomplete bond due to loss of water from mortar to masonry unit. — If a highly absorbent masonry unit is used, and it has not been properly wetted before laying, the mortar may lose sufficient water to the masonry unit almost immediately after it is spread, so that the workability of the mortar is lost and it becomes impossible for the mason to bring the next unit into complete contact with the mortar. This stiffening of the mortar results in cracks which increase in size and extent under freezing and thawing in the presence of moisture. The bond between mortar and the masonry unit on which it is spread is also incomplete, due to loss of water from the mortar.

Authorities differ as to what takes place at the juncture of mortar and brick. However, if cracks occur, they will increase in size under the action of weathering. From the data available it appears that separation cracks may be largely eliminated through the proper selection of masonry materials. The requirements are:

(I) Masonry units having a low rate of absorption when laid — the absorption of high absorption units may be controlled by wetting prior to laying.
(II) Mortar of high water retentivity.

It should be emphasized, however, that good workmanship is essential to watertight masonry, and that no materials regardless of how well suited they may be for the purpose, will produce weather resistant masonry if poor workmanship is employed in the construction.

## 220. MORTAR PROPERTIES

Numerous investigators have listed the desirable properties of masonry mortars. There is substantial agreement as to what these properties are. However, their relative importance is highly controversial. The competition between the producers of mortar materials is very keen, and it is to be expected that each manufacturer will emphasize the importance of the particular mortar properties to which his own material contributes. Frequently one property of mortar is obtained only at the expense of another. Some plasticizers, for instance, which increase the workability of mortar also tend to reduce its strength.

Much has been written during the past ten years on the question of mortar with particular emphasis on mortar properties. However, due to the divergence of opinion expressed in many of these writings, the result has been to confuse the consumer rather than to aid him in developing a proper specification.

The mass of advertising literature devoted to mortar and mortar ingredients with which the public has been flooded, also tends to place an undue emphasis on the importance of mortar and to create the impression that mortar alone is responsible for satisfactory or unsatisfactory masonry construction, and that if an ideal mortar can be obtained other more important factors may be ignored with safety.

Experience in building clay products structures indicates that good results can be obtained with mortars produced from standard materials which are obtainable generally without paying any premium for special ingredients, and that in a great majority of cases any one of a number of mortar mixtures can be used with entire satisfaction. In such instances, the cost of the mortar might well determine the selection.

## 221. WORKABILITY AND WATER RETENTIVITY

Workability is an essential property of any mortar for masonry construction, since it is only through this property that the mortar can be brought into intimate and complete contact with the masonry units and thereby incorporate the properties of the

mortar into the masonry. Workability is closely related to water retentivity and as a general rule, the higher the water retentivity of a mortar, the greater its workability. Water retentivity is measured by the flow after suction test. This test is described in Federal Specification SS C181b which contains the requirement that "Standard mortar after suction for 60 seconds shall have a flow greater than 65% of that immediately after mixing." Where high strength mortars are required as in brick sewer construction or in reinforced brick masonry, particularly in contact with earth, it may be necessary to reduce this requirement for water retentivity. However, satisfactory mortars are available for most uses which have flows after suction of 65% or more and this requirement should, as a rule be included in the specification.

## 222. BOND STRENGTH OR ADHESION

All authorities agree on the importance of the completeness and durability of the bond between the mortar and brick. The strength of this bond depends primarily upon the penetrability (rate of absorption) of the brick when laid and upon the flow, water retentivity and strength of the mortar. Fig. 5 shows the relation between penetrability of brick and tensile bond strength for three mortar mixtures of identical flows. Penetrability of brick is given as the grams of water absorbed by the flat side of the brick

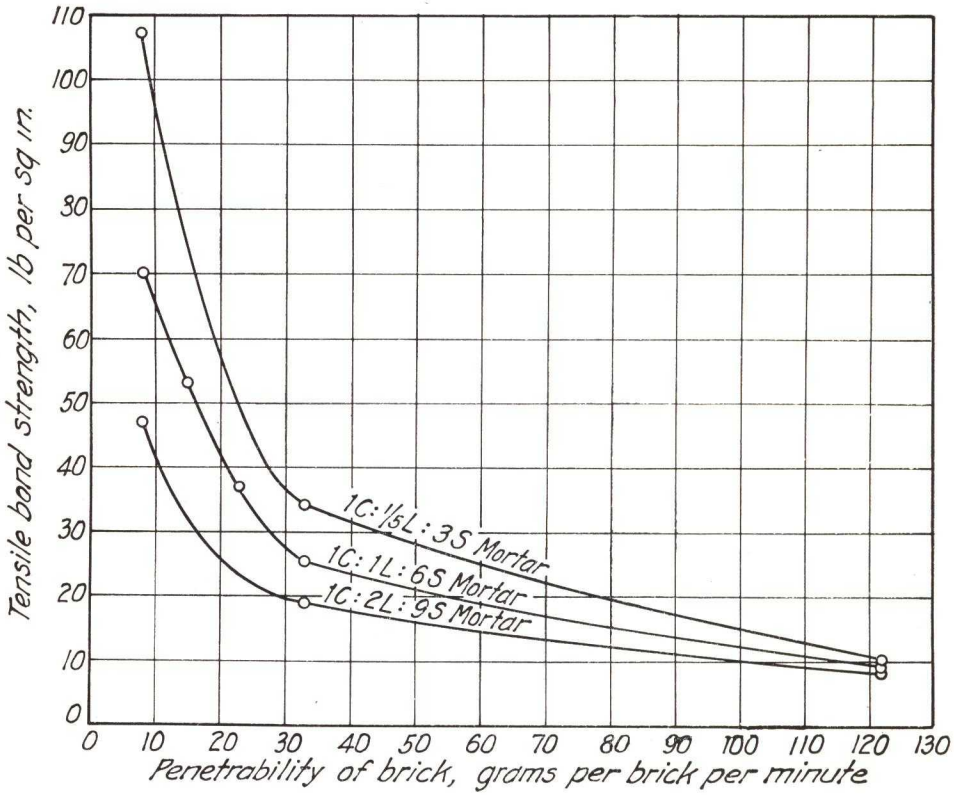



*Figure 5*

*Showing relation between penetrability of brick and tensile bond strength of three mortar mixtures.*

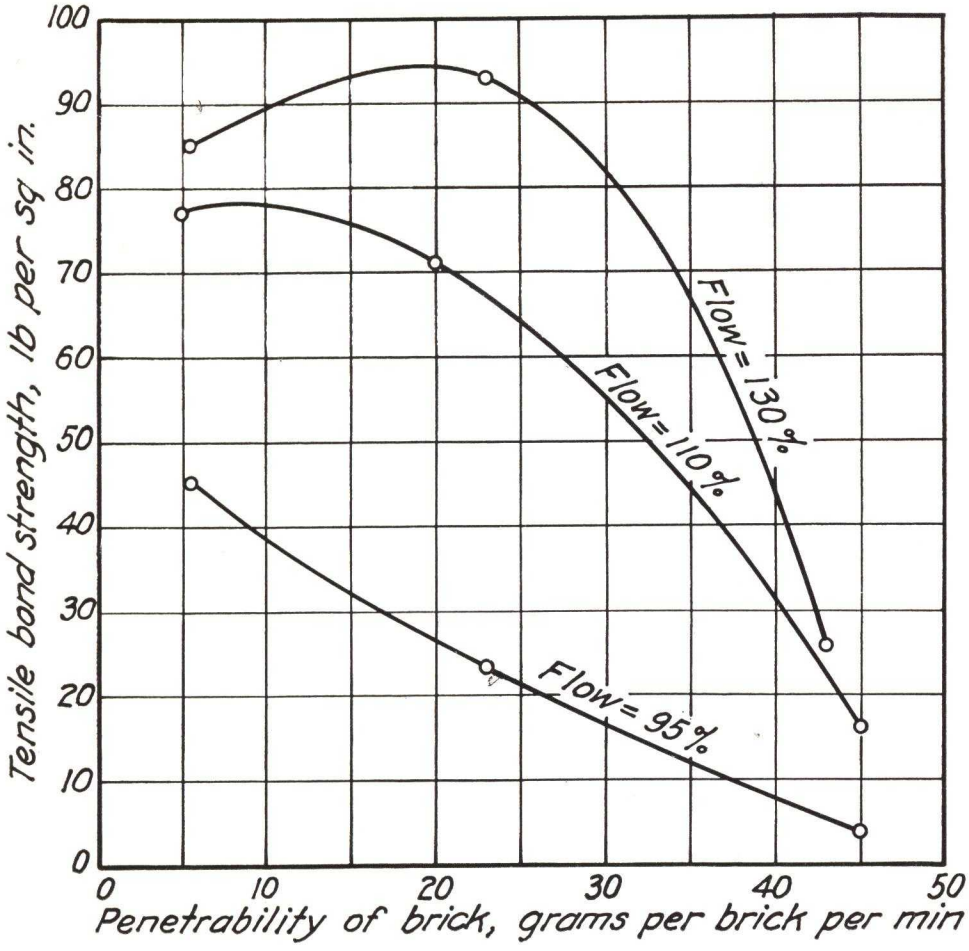



*Figure 6*

*Showing relation of tensile bond strength to penetrability of brick for three mortars having different flows.*

when immersed in ⅛″ of water for one minute. The data plotted in Fig. 5 indicate that for the three mortar mixtures considered, tensile bond strength increased as penetrability of brick decreased. Fig. 6 shows the relation of tensile bond strength to penetrability of brick for three mortars having flows of 95%, 110% and 130%. This is a composite chart, the mortars represented by the different curves are not all of the same composition, consequently, the chart does not show the relation of tensile bond strength to flow; however, it does indicate that for high flow mortars the optimum penetrability of the brick from the standpoint of tensile bond strength is approximately 20 grams per minute. Brick having penetrabilities when laid below 20 grams per minute will tend to float with high flow mortars. Consequently, it may be stated that for highest tensile bond strength the penetrability of brick when laid should be as low as possible without floating. Fig. 7 shows the relation between tensile bond strength and compressive strength of 2″ mortar cubes.

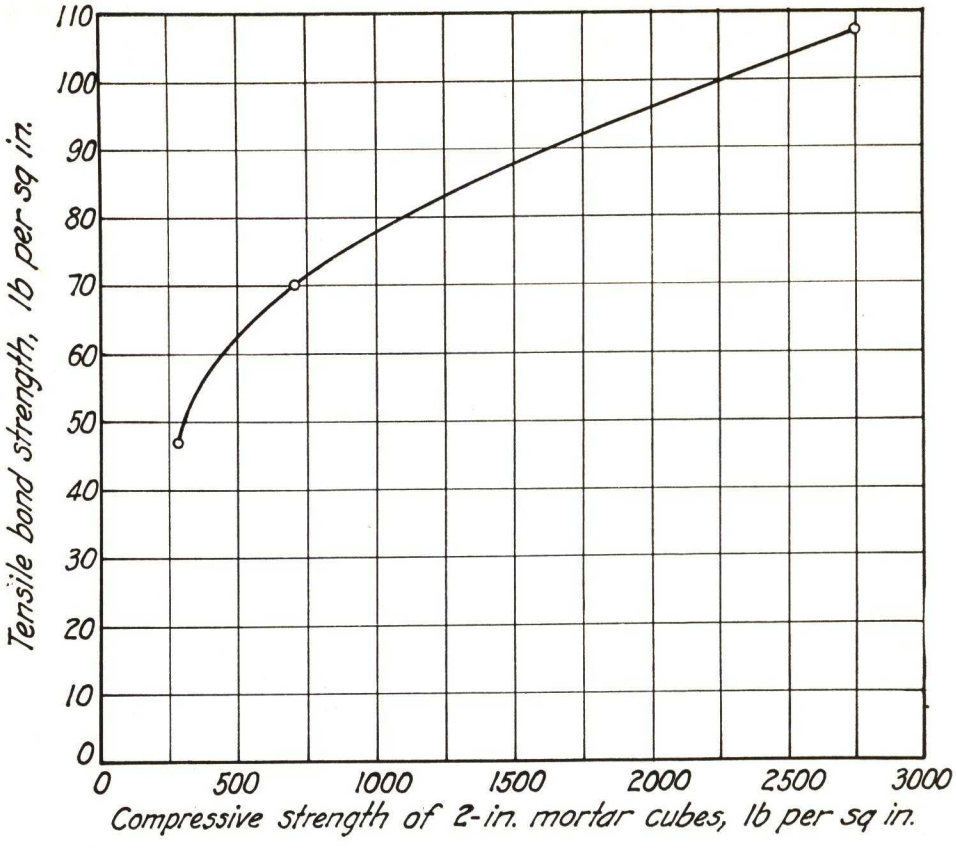



*Figure 7*
*Showing relative compressive strength of mortar and tensile bond strength.*

From the standpoint of weather resistance, the extent or completeness of the bond between mortar and masonry units is probably more important than the strength of the bond except as strength may be related to durability. However, strength of bond is an important factor, particularly in the lateral strength of masonry and reasonable strength can be obtained with many mortars in conjunction with a good extent of bond.

The bond of mortar is impaired by poor water retentivity which tends to make the bond "patchy". Such a bond may be completely destroyed within a short time by freezing and thawing in the presence of moisture.

### 223. WEATHER RESISTANCE OR DURABILITY

The importance of this property of mortar will depend upon the exposure of the masonry and the type of construction in which it is to be used. Weatherability of mortar is related to its strength; however, its resistance to disintegration as a part of a masonry wall cannot always be judged by either the strength or resistance to freezing and thawing of the mortar in cubes or bars, apart from the masonry unit. For relatively dry masonry which resists the penetration of excessive moisture, the resistance of mortar to alternate freezing and thawing in the presence of moisture is of little importance.

However, in structures where the masonry will be at least partially saturated on frequent occasions, due either to its location or design, frost resistance mortar is important. Experience is, perhaps, the best guide in judging this property for any mortar in a given locality. In the absence of information on the performance of a mortar, a minimum strength requirement of 600 lbs. per sq. in. for 2″ cubes at the age of 7 days (1 day storage in moist air, 6 days in water) is recommended for mortar in masonry subjected to severe exposure.

## 224. STRENGTH

The strength of mortar affects both the compressive and transverse strength of the masonry and the importance of this property, except as it may also relate to the durability of mortar, depends primarily on the type of construction in which the mortar will be used. Except in engineering structures, high compressive strength of masonry is rarely required. Tests to determine the compressive strength of masonry indicate that for brick having compressive strength of approximately 3500 lbs. per sq. in., the strength of brick masonry using different mortars increases approximately as the cube root of the strength of the mortar, instead of directly with the mortar strength. For higher brick strengths the differences in masonry strength caused by changes in mortar strength are greater. The following data were reported in Bureau of Standards Research, Paper 108.

| MORTAR | | MASONRY STRENGTH, LB. PER SQ. IN. | |
|---|---|---|---|
| Mix | Strength* | 3540#/sq. in. brick | 8600#/sq. in. brick |
| 1:1:6 | 1100 | 945 | 1840 |
| 1:1/10:3 | 3260 | 1145 | 2710 |

*Mortar compressive strength is 60 day strength of 2″ cubes cured in water.

Mortar strength probably has a greater effect upon transverse strength of masonry than upon the compressive strength, although little data are available on the subject. Transverse strength of masonry is affected both by workmanship and the absorption of the masonry unit when laid. Consequently, unless these factors are constant, a comparison of the lateral strengths of walls laid up with different mortars may be misleading. From the limited data available, the transverse strength of brick walls with good workmanship (flat bed joints) and low absorption units when laid increases with the mortar strength, provided the bond between mortar and brick is practically complete. The following table from Bureau of Standards, Report BMS5 indicates the possible increase in wall strength, due to increased mortar strength:

**Transverse Strength of 8″ Solid Brick Walls**

EQUIVALENT UNIFORM LOADS LB./SQ. FT., SPAN 7′-6″

| BRICK | | MORTAR | | Transverse Strength of Masonry Wall**** |
|---|---|---|---|---|
| No. | Absorption* grams/brick | Mix.** | Comp. Str.*** lb./sq. in. | lb./sq. ft. |
| 1 | 8 | 1-¼-3 | 3220 | 125 |
| 2 | 11 | 1- 1-6 | 640 | 82 |

* One minute partial immersion—as laid—Brick No. 2 were laid damp.
** Proportions, cement, lime putty, sand by volume.
*** Two Inch cubes, age 28 days, water storage.
****Average of three tests.

## 225. EFFLORESCENCE

Mortars which contain an excess of soluble salts will contribute to efflorescence of the masonry. Efflorescence can only occur as water penetrates the masonry, dissolves the salts and upon evaporation deposits them on the face of the wall. The surest preventative of efflorescence is to keep the masonry dry. However, since under normal conditions, some moisture will enter the wall, it is desirable to use a mortar which contains a minimum amount of soluble salts. Numerous attempts have been made to develop admixtures for mortar which would combine with the soluble salts present to form insoluble compounds. Barium Carbonate seemed to offer possibilities as such an admixture; however, investigations to date (1939) indicate that admixtures are only slightly, if at all, effective in reducing efflorescence, and that they cannot be relied upon as a preventative if both moisture and soluble salts are present in the masonry.

Various methods have been devised to test mortars for efflorescence. One is to prepare a small slab of the mortar and to subject it to a test similar to the wick test for brick described in Sec. 124, Chapter 1. This test will indicate the tendency of the mortar to effloresce; however, if soluble salts are present, it will not disclose their source. A more complete test is to place bricks which have been found to be free from efflorescence in pans containing:

(a) ½″ of the water to be used in mixing the mortar.

(b) ½″ of distilled water to which a small amount of the mortar sand has been added.

(c) ½″ of distilled water with a small amount of lime added.

(d) ½″ of distilled water with a small amount of cement added.

(e) ½″ of the water to be used in mixing the mortar to which small portions of sand, lime and cement have been added.

If efflorescence appears on any of the brick it will be due to one of the mortar materials or possibly (e) a combination of all of them.

## 226. PERMEABILITY

L. W. Burridge of the Clay Products Technical Bureau of Great Britain, and the British Building Research Station recommend mortar of a fair degree of porosity on the theory that such mortar promotes drying of the masonry by providing a path of escape for the moisture through the mortar joint. According to this theory, if the brick are impervious, there is all the more reason for having a porous mortar, especially for pointing. Field observations both here and abroad, tend to support this view. If there are openings in the joints and if both brick and mortar are impervious, the wall will accumulate water far more rapidly than it can exude it and prolonged saturation takes place. It is also argued that if moisture escapes through the mortar joint, soluble salts which may be present in the mortar, will be deposited on the face of the mortar joint where they are less objectionable than on the brick.

While reasonable porosity is admittedly desirable in a mortar, it should not be sufficiently porous to permit the penetration of excessive moisture into the masonry. Mortars having absorptions of from 6% to 12% by 24-hr. cold immersion are recommended for exposed masonry above grade.

For underground structures such as sewers, basement walls and retaining walls, the mortar should be as impermeable as possible, since the percolation of water through

the mortar will tend to leech out the cementitious materials and eventually cause disintegration.

## 227. VOLUME CHANGE

Cracks in masonry ranging all the way from structural cracks to unfilled mortar joints, have been attributed to volume changes in the mortar and from the importance attached to this property by much of the current literature, the impression prevails in some quarters that no masonry is safe from the "disastrous" effects of volume change, unless special ingredients are added to the mortar to control it. There are volume changes in all mortars, due to temperature variation and wetting and drying and a shrinkage both during and after hardening. These changes are comparable to similar changes of other standard building materials and, undoubtedly, are not responsible for many of the observed cracks between mortar and masonry units which have been attributed to volume change. There is much evidence both from the field and from the laboratory to support the claim that volume change is of minor importance from the standpoint of moisture penetration, weatherability, or strength of masonry.

## 228. OTHER PROPERTIES

Extensibility or flexibility, rate of stiffening, staining and fading, are other properties of mortar that have been listed by some writers. These properties are either reasonably constant for all mortars (Extensibility), are closely related to other properties which have been discussed (rate of stiffening) or are of minor importance in connection with clay product masonry and will not be included in this discussion.

## 229. MORTAR ADAPTED TO MASONRY UNITS

Several investigators have suggested that mortars should be adapted to the masonry units with which they will be used. There is much merit in this suggestion; however, the range of mortars suitable for use with brick of various absorptions and rates of absorption is by no means as great as the range of absorptions of brick. A highly impervious masonry unit tends to float when placed on a mortar bed and, consequently, a mortar suitable for this type of unit should have a low flow, but at the same time should be workable. F. O. Anderegg recommends, "Watertight Brick Masonry", The Architectural Record, September, 1931: "With brick of lower absorption rates mortars of low water content are recommended; limes of 'low yield' are helpful. Moreover, the mortar should be quite workable in order to aid in securing contact with the brick, and the rate at which moisture leaves the mortar should also be controlled. An adequately distributed stearate waterproofer seems necessary for such control." However, for the higher absorptive units, a mortar of higher flow would not be objectionable and if in certain localities it is necessary to pay a premium for a workable low water content mortar, a cheaper mortar might be used for more absorbent units.

## 230. SPECIFICATION FOR MORTAR FOR CLAY PRODUCTS MASONRY

### 1. PROPORTION

All mortar for ..... (insert use) shall consist of a mixture of ..... (specify mix) portland cement, (.....) lime putty and (.....) sand proportioned by volume, mixed with a maximum amount of water consistent with satisfactory use.

## 2. QUALITY OF MATERIALS

### (a) Portland Cement

Portland cement shall conform to the Standard Specifications for Portland Cement, A.S.T.M. C9-38.

### (b) Lime Putty

Lime putty may be made from either quick lime or hydrated lime. If quick lime is used, it shall conform to the Standard Specifications for Quick Lime for Structural Purposes, A.S.T.M. C5-26, and shall be thoroughly slacked in accordance with the directions given in the appendix to specification C5-26 and the recommendations of the manufacturer. If hydrated lime is used, it shall conform to the standard specification for Hydrated Lime for Structural Purposes, A.S.T.M. C6-31, and shall be soaked at least 12 hr. prior to use. Lime putty shall have a plasticity figure of not less than 200, determined by the procedure outlined in A.S.T.M. Specification C6-31 for Finishing Lime.

### (c) Aggregate (sand)

Aggregate shall consist of fine granular material composed of hard, strong, durable mineral particles which are free from injurious amounts of saline, alkaline, organic or other deleterious substances. Aggregate shall be so graded that when tested in accordance with the Standard Method of Test for Sieve Analysis of Aggregates for Concrete, A.S.T.M. C41-36, the percentages by weight passing each sieve shall fall within the limits specified in the following table:

| SIEVE | PERCENTAGE BY WEIGHT PASSING EACH SIEVE | |
|---|---|---|
| | Minimum | Maximum |
| No. 4 | 100 | 100 |
| No. 8 | 96 | 100 |
| No. 16 | 70 | 90 |
| No. 30 | 40 | 70 |
| No. 50 | 15 | 35 |
| No. 100 | 5 | 15 |
| No. 200 | 0 | 5 |

(Designer should change percentages in above table to conform to local material if possible, but range (min. to max.) should remain fixed.)

Aggregate shall be of such quality that when subjected to the mortar strength test in accordance with Standard Method of Test for Structural Strength of Fine Aggregate Using Constant Water-Cement-Ratio, A.S.T.M. C87-36, it shall have a compressive strength at ages of 7 and 28 days of not less than 95% of that developed by a mortar of the same water-cement-ratio and consistency made with the same cement and graded Ottawa sand having a fineness modulus of $2.40 \pm 0.10$.

Aggregate subjected to the Standard Method of Test for Organic Impurities in Sands for Concrete, A.S.T.M. C40-33, shall be rejected if it produces a color darker than the standard, unless it passes the mortar strength test.

The maximum of coal and lignite as determined by Tentative Methods of Test for Coal and Lignite in Sand A.S.T.M. C-123, shall not exceed 0.25% by weight and the maximum of clay lumps as determined by Tentative Method of Test for Clay Lumps in Aggregate A.S.T.M. C-142, shall not exceed 1% by weight.

### (d) Water

Only clean fresh water, free from acids, alkalis or organic matter shall be used in mixing mortar.

**(e) Integral Water Proofing Compounds or other Admixtures**

Integral water proofing compounds not definitely mentioned in the specifications shall not be used in mortar without approval in writing from the purchaser.

**(f) Mortar Colors**

Only pure mineral mortar colors shall be used. The brand and quality of such coloring, and the amount to be used shall, unless definitely stipulated in the specifications, be approved in writing by the purchaser.

**(g) Anti-Freeze Compounds**

No anti-freeze liquid, salts or other substances shall be used in the mortar to lower the freezing point.

**3. PROPORTIONING AND MIXING**

(a) Measurement of Materials. The method of measuring materials on the job shall be such that the specified proportions of the mortar materials can be controlled and accurately maintained during the entire progress of the work.

(b) Mixing mortars. All mixing of mortar shall preferably be done in a mechanically operated batch mixer of the drum type. The use of a continuous mortar mixer is not recommended and will be permitted only when approved by the purchaser. Hand mixing will be permitted on small jobs, provided that the quantity of materials and water is controlled and accurately maintained and provided further that the hand mixing is done in a manner meeting with the approval of the purchaser. When a batch mixer is used, the mortar shall be mixed for a period of at least three minutes after all materials for a batch are in the drum. The drum must be completely emptied before the succeeding batch is placed therein.

**4. WATER RETENTIVITY**

Mortar as delivered to the mason shall have a "Flow after suction for 1 minute" as outlined in Federal Specification SS C-181b of not less than 65%.
(If facilities for making Water Retentivity test are not available Sec. 4 should not be included in the specification.)

**5. TESTS OF MORTAR**

At least three 2″ test cubes of mortar shall be made and tested for each lot of 50,000 brick equivalent or less. Cube test specimens shall be made from mortar actually being used or proposed to be used on the job.

Cubes shall be molded either at the job or in the laboratory in accordance with the requirements of Section 12 of Tentative Method of Test for Compressive Strength of Portland Cement Mortars, A.S.T.M. C109-37T. If molded in the laboratory, the mortar shall be stored and delivered in air-tight metal containers and not more than one-half hour shall elapse between the time of mixing and the time of molding the cubes.

Cubes shall be tested at the age of 7 days and shall have an average compressive strength of not less than the following:

| MORTAR MIX* | MINIMUM COMPRESSIVE STRENGTH lbs./sq. in. |
|---|---|
| 1-1-6 | 400 |
| 1-¼-3 | 1500 |
| 1-2-9 | 150 |

*Proportions by volume; cement, lime putty, sand.

## 6. COST OF TEST

The supplier shall provide the materials for the mortar specimens free of charge. Unless otherwise specified with the contract and order, the cost of the tests shall be borne as follows:

(a) By the Supplier in the event of results showing that the mortar does not comply with the specification.

(b) By the Purchaser in the event of the results showing that the mortar complies with the specification.

## 7. ACCEPTANCE AND REJECTION

Mortar shall be rejected for failure to meet any of the requirements of the specification. In case of failure to comply with the specification, the supplier shall make such change in ingredients or proportioning of the mortar as will ensure compliance.

## 231. SUMMARY

From a review of the technical literature on masonry and masonry mortars and the reports of field experience and investigations, the following statements appear justified:

1. A "good" mortar is one that will adhere to masonry units and when used properly with satisfactory units will produce weather resistant, durable masonry of the required strength and desired appearance.

2. For exposed masonry, impermeability (water-tightness) is of primary importance, particularly in the presence of freezing and thawing.

3. All masonry mortars should be workable, and have a maximum flow consistent with satisfactory use.

4. A high degree of water retentivity is desirable in most mortars. Water retentivity is related to workability. High retentivity and high workability are found in the same mortars.

5. The use of masonry units and mortar which resist laboratory freezing and thawing tests will not insure durable masonry unless the masonry resists water penetration over a long period of time. Materials rating "poor" in laboratory freezing and thawing tests may be durable in dry masonry, especially where not subjected to severe exposure.

6. Compressive strength of masonry is related to brick strength, mortar strength and method of laying. The compressive strength of most brick masonry is in excess of the requirements.

7. Principal factors controlling the transverse strength of brick masonry are mortar strength and water retentivity, penetrability of brick and workmanship.

8. Mortar may mar the appearance of brick masonry by:

(a) Bleeding: loss of water during construction which stains the masonry units.

(b) Providing a source of soluble salts which appear as efflorescence.

(c) Erosion under the action of weather.

9. Mortar, particularly for use with dense masonry units, in exterior walls above grade, should be reasonably porous (6% to 12% absorption, 24 hr. cold immersion) in

order to permit the escape of moisture from the wall through the mortar joint, and to retard the deposit of soluble salts originally in the mortar on the face of the masonry unit.

10. Volume changes due to alternate wetting and drying as well as shrinkage during and after set, which exist in all mortars, are of minor importance for mortars of the approximate proportions: 1 part cementitious material to 3 parts aggregate by volume. (This statement is controversial.)

## 232. RECOMMENDATIONS

It should be borne in mind that the suitability of a mortar for any purpose depends upon its properties and that desirable properties are to be found in mortars consisting of a wide variety of ingredients and proportions. However, until standard methods have been developed for determining and evaluating mortar properties in relation to the performance of the mortar in masonry construction, the designer must judge the suitability of various mortar mixtures by their records of performance and the available information regarding their properties.

It is believed that the mortar mixtures recommended are suitable for the uses indicated, but it goes without saying that the list is not complete and that there are many other mortars, particularly masonry cement mortars, which are equally satisfactory.

The classification of uses for the purpose of general recommendations, must of necessity be limited and due to the variation in exposures and conditions of use existing in different localities and in different structures, it may prove desirable to expand or restrict the uses of the mortar mixtures listed in certain districts.

Recommendations are in general applicable to exposures and construction practices prevailing in the northeastern quarter of the United States. They are based upon field experience and the technical data now available. However, it would be presumptuous to assume that this publication constitutes the last word on the subject and as additional data are developed through researches now in progress, further recommendations may be justified.

All of the ingredients of the mortar mixes suggested should conform to A.S.T.M. or other standard specifications. Lime putty refers to well slaked quick lime or hydrated lime that has been soaked prior to use.

All proportions are by volume.

**1. 1 cement: 1 lime putty: 5 to 6 sand.**

Suitable for general use above grade and recommended specifically for parapet walls, chimneys and exterior walls subject to severe exposures, also for structural clay tile construction.

**2. 1 cement: 2 lime putty: 7 to 9 sand.**

Suitable for non-load-bearing walls either interior or exterior not subjected to severe exposures, also load-bearing walls in which unit compressive stress is not excessive. (Building code requirements may be used as a guide.)

**3. 1 cement: ¼ lime: 2½ to 3 sand.**

Suitable for general use and recommended specifically for reinforced brick masonry and plain masonry below grade or in contact with earth, such as foundations, retaining walls, walks, sewers, manholes, and catchbasins.

The properties of masonry cements vary widely and no attempt has been made to include them in the above list. However, experience and test data indicate beyond doubt that certain of the masonry cements produce mortars that are suitable for general use.

## 233. SPECIFICATIONS

Present day common practice in specifying mortar is on a proportion basis, usually definite parts by volume of cement, lime and sand. Formerly the proportion basis was also used in specifying concrete. However, in recent years the tendency has been to specify the quality of the ingredients of concrete and to write a specification for the concrete itself based upon its properties, permitting the builder a wide latitude in the design of the concrete mix to obtain the required properties. Most designers agree that a property specification, wherever it is possible to use it, is the better specification since it does not limit the builder in his methods of construction. A property specification, however, presupposes that tests of the properties specified have been developed and that when the material is submitted for approval it can be judged on the basis of these tests. A.S.T.M. Committee C-12 is now (1939) working on the development of such tests for mortar and after the test procedures have been established and adequate measures of the various properties have been determined, a property specification will be developed.

The following typical specification is a proportion specification to which certain property requirements have been added.

It is believed that the seller will find no difficulty in obtaining the required properties with the mortar mixtures suggested, provided suitable ingredients are used and proper care is taken in proportioning and mixing.

# CHAPTER 3

# BRICK MASONRY

## 301. PROPERTIES

While the chemical and physical properties of individual brick are important, the properties of brick masonry are those with which we are most concerned. Properties of individual brick do not accurately measure, but only indicate the quality of brick masonry built therefrom.

The principal properties or requisites of brick masonry are compressive and transverse strengths, resistance to fire, weathering, moisture penetration, heat transmission and sound transmission. Pleasing appearance is frequently an important consideration, particularly in exposed surfaces.

The ability of brick masonry to meet any or all of these requirements has been studied for many years in laboratories and by field observation. The data thus obtained have been given in reports of the National Bureau of Standards and many other testing laboratories, technical articles in various engineering and architectural publications, as well as literature of the Structural Clay Products Institute. For the purpose of this book, reports of tests and the conclusions will be condensed and only the fundamental data will be given for consideration in the design of brick masonry construction.

## 302. COMPRESSIVE STRENGTH

Compressive strength is not a fixed or constant quantity, but varies and is influenced by these principal factors; compressive and shearing strength of the brick unit, strength of mortar and quality of workmanship. Other factors affecting the compressive strength of brick masonry are: type of construction, regularity in size, and shape of brick and curing conditions.

As one would assume, a stronger brick, stronger mortar and better quality of workmanship will produce stronger brick masonry. All other factors being equal, the brick most regular in size and with plane, parallel faces, will give the highest masonry strength. Within the limits of workability, narrower mortar joints contribute to stronger masonry.

In walls built with inferior workmanship, when the vertical joints between wythes are not completely filled with mortar, or when the weaker mortars are used, masonry bonding courses at frequent intervals will increase the strength of the wall. In brick masonry with all joints completely filled with strong mortar, the brick header or other masonry bond is not so important. In fact, tests have proven that very satisfactory walls can be built without brick header bonds by filling the interior vertical joints between wythes with grout.

The following data, including the compressive strength of a number of brick walls, have been copied from the National Bureau of Standards' Research Paper No. 108, "Compressive Strength of Clay Brick Walls", by A. H. Stang, D. E. Parsons, and J. W. McBurney. Ten types of walls, 168 in all, were constructed and tested. Sectional views of the wall types are shown in Figure 8. The walls were six feet long and about 9 feet high. These tests indicate the effect of such variables as workmanship, strength of brick, kind of mortar and type of construction.

Four kinds of brick were used in building the specimens for this investigation. For convenience these brick were designated by the name of the region in which they were made and are described as follows:

Chicago brick were made from surface clay and formed by the end-cut, double-column, stiff-mud process. They are rather irregular in shape and contain lime nodules.

Detroit brick were formed by the soft-mud process from surface clay. Like many soft-mud brick, they were formed with a frog or depression in one face approximately 0.4 inch deep. These Detroit brick resemble the soft-mud brick from the Cleveland (Ohio) region.

Mississippi brick were surface-clay brick formed by the dry-press process. Regularity of size and shape was the outstanding characteristic of these specimens.

New England brick were formed by the soft-mud process from surface clay and were "sand struck." The specimens contained a shallow frog or depression, were very hard burned, and rather irregular in size and shape.

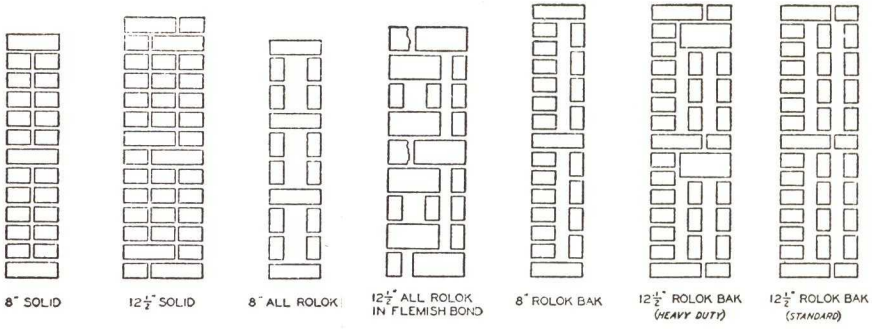



*Figure 8*
*Sectional views of masonry walls.*

The 8″ and the 12″ solid walls were laid in common American bond, having headers each sixth course. The brick in these walls were laid flat, but none of the solid walls had header courses at the top or bottom.

The hollow walls had header courses at the top and bottom. In the all-rolok walls the stretchers are laid on edge. Every third course is a header course with the brick flat. In the 8″ all-rolok wall, the headers are side by side and extend completely through the wall. In the 12″ wall of this type, the center wythe of brick on edge is not centered, but is lined up with the outside headers. The header courses are of "basket weave", in which two header brick joining the face wythe with the central wythe alternate with two header brick joining the back wythe with the central wythe, the spaces opposite each pair of headers being occupied by a single stretcher.

In all-rolok in Flemish bond walls, all the brick are on edge laid in Flemish bond for the outside 8″ width. For the 12″ wall, a wythe of stretchers is added, three courses high. The appearance of the front of the wall in the next course is still of the Flemish bond, but the headers of this course are bats and the backing is a continuous course of rowlock headers.

The exterior 4″ thickness of the rolok-bak walls is laid with the brick flat and the backing is laid of brick on edge. On the exterior, therefore, the brickwork has the usual appearance of ordinary brickwork having headers each seventh course. Four courses of brick on edge bring the backing courses to the same height as that of the six flat courses of the face. The 12″ rolok-bak wall is the heavy-duty type. In this type, the fourth backing course consists of rolok-headers. Two 12″ rolok-bak walls, standard type, were included in this investigation. The standard type differs from the heavy-duty type by having the four backing courses all stretchers, the flat header course being of "basket weave".

The economy wall is essentially a 4″ wall with pilasters 8″ wide and 4″ thick built into the wall at intervals of about 5½ stretcher lengths. The pilasters were tied to the 4″ wythe with headers each sixth course. All brick were laid flat. The "economy" walls

of this investigation were plastered on the back between the pilasters with the same kind of mortar as that used in laying the brick, since this method of construction is recommended for weatherproofing the single wythe.

The results of the tests of the single brick are given in the following table. Each average value, except for shearing strength, is the average result from 50 tests.

**AVERAGE PHYSICAL PROPERTIES OF THE BRICK**

| Kind of Brick | Absorption | | Modulus of Rupture | | Compressive Strength | | Tensile Strgth. | Shearing Strgth. |
|---|---|---|---|---|---|---|---|---|
| | 5 hr. boiling | 48 hr. cold | flatwise | edgewise | flatwise | edgewise | lbs. per sq. in. | lbs. per sq. in. |
| | Per cent | | Lbs. per sq. in. | | Lbs. per sq. in. | | | |
| Chicago | 16.5 | 11.7 | 1225 | 1340 | 3280 | 3350 | 417 | 1100 |
| Detroit | 22.3 | 20.7 | 670 | 680 | 3540 | 3270 | 222 | 1165 |
| Mississippi | 21.7 | 16.7 | 820 | 760 | 3410 | 3625 | 317 | 1590 |
| New England | 9.2 | 6.9 | 1550 | 1640 | 8600 | 11470 | 601 | 3550 |

The mortar mixes represent certain commonly used volume proportions. Measurements by volume, however, would have resulted in wider variations in the mortar compositions than seemed desirable so that equivalent proportions by weight were used, assuming that 1 cubic foot of lime weighs 40 lbs. and 1 cubic foot of cement weighs 94 lbs. The weight of the dry materials in a cubic foot, loose measure, of the damp sand used on this work was determined by preliminary tests to be about 73 lbs. Since the weight of a cubic foot of damp sand, loose measure, varies with the moisture content, the moisture in a sample of sand was determined each day during the construction of the walls and the weight necessary to make the desired amount of dry sand was computed. This value was used in proportioning the mortar for the day. Water was added to give the consistency desired by the mason and the amount of water recorded. All the mortar used was proportioned by these equivalent weights.

The mortar mixtures were cement-lime and cement mortars. They were in conformity with the mortar recommended for solid walls by the Building Code Committee of the Department of Commerce. The mixtures were as follows:

**Cement-lime mortar.** — By volume, 1 part of Portland cement, 1 part of hydrated lime, to 6 parts of damp sand; weight equivalents, 94 pounds of cement, 40 pounds of lime, to 440 pounds of dry sand.

**Cement mortar.** — By volume, 1 part of Portland cement to 3 parts of damp sand (hydrated lime equal to 10% of the volume of the cement was added); weight equivalents, 94 pounds of cement, 4 pounds of lime, to 220 pounds of dry sand.

The average results of the tests of the mortar specimens are given in the following table:

**COMPRESSIVE STRENGTH OF MORTAR SPECIMENS**

| Mortar | Proportions (By Volume) | STRENGTH* | |
|---|---|---|---|
| | | Cured Wet lb./sq. inch | Cured Dry lb./sq. inch |
| Cement-lime | 1C:1L:6S | 1100 | 750 |
| Cement | 1C:1/10L:3S | 3260 | 1950 |

*Cylinders 2 inches in diameter, 4 inches long tested at 57 to 62 days.

Table No. 11 gives the compressive strength of a series of brick walls built according to best recommended practice. The workmanship on these walls was inspected. The

brick mason was instructed to use smooth mortar beds and fill all vertical joints completely.

## TABLE NO. 11

## RESULTS OF COMPRESSIVE TESTS OF BRICK WALLS

### Series "A"

### (Inspected Workmanship)

| Type of Wall | Mortar | Kind of Brick | Wall Strength* lb./sq. inch | | | |
|---|---|---|---|---|---|---|
| | | | a | b | c | Average |
| 8″ Solid | Cement-Lime | Detroit | 1060 | 905 | 760 | 910 |
| | | Mississippi | 965 | 1175 | 1340 | 1160 |
| | | New England | 1785 | 1875 | 1710 | 1790 |
| | Cement | Chicago | 800 | 870 | 965 | 880 |
| | | Chicago | 885 | 925 | 755 | 855 |
| | | Detroit | 1305 | 895 | 1050 | 1080 |
| | | Detroit | 1055 | ..... | ..... | 1055 |
| | | Mississippi | 1335 | 1405 | 1405 | 1380 |
| | | New England | 2040 | 2950 | 2915 | 2635 |
| 12″ solid | Cement-Lime | Detroit | 1050 | 940 | 965 | 985 |
| | | Mississippi | 1375 | 1305 | 1225 | 1300 |
| | | New England | 1815 | 1990 | 1855 | 1890 |
| | Cement | Chicago | 1165 | ..... | ..... | 1165 |
| | | Chicago | 890 | 1080 | ..... | 985 |
| | | Chicago | 1200 | ..... | ..... | 1200 |
| | | Detroit | 1165 | 1345 | 1115 | 1210 |
| | | Detroit | 1040 | 1165 | ..... | 1105 |
| | | Mississippi | 1710 | 1855 | 1350 | 1640 |
| | | Mississippi | 1305 | 1875 | ..... | 1590 |
| | | Mississippi | 1465 | ..... | ..... | 1465 |
| | | New England | 2440 | 3230 | 2695 | 2790 |
| | | New England | 1750 | 3270 | ..... | 2510 |
| | | New England | 2720 | ..... | ..... | 2720 |
| 8″ All-Rolok | Cement-Lime | Mississippi | 745 | 655 | 890 | 765 |
| | | New England | 1055 | 780 | 795 | 875 |
| | Cement | Mississippi | 850 | 735 | 1175 | 920 |
| | | New England | 1005 | 1065 | 795 | 955 |
| 12″ All-Rolok | Cement-Lime | Mississippi | 645 | 725 | 750 | 705 |
| | | New England | 805 | 785 | 640 | 740 |
| | Cement | Mississippi | 795 | 745 | 870 | 800 |
| | | New England | 775 | 935 | 895 | 870 |

*Tested at 57 to 62 days.

## TABLE NO. 11 — Continued

## RESULTS OF COMPRESSIVE TESTS OF BRICK WALLS

### Series "A"

### (Inspected Workmanship)

| Type of Wall | Mortar | Kind of Brick | Wall Strength* lb./sq. inch | | | |
|---|---|---|---|---|---|---|
| | | | a | b | c | Average |
| 8″ All-Rolok Flemish Bond | Cement-Lime | Mississippi | 800 | 750 | 695 | 750 |
| | | New England | 555 | 745 | 610 | 640 |
| | Cement | Mississippi | 825 | 720 | 855 | 800 |
| | | New England | 680 | 925 | 720 | 775 |
| 12″ All-Rolok in Flemish Bond | Cement-Lime | Mississippi | 570 | 640 | 810 | 675 |
| | | New England | 890 | 900 | 700 | 830 |
| | Cement | Mississippi | 550 | 660 | 920 | 710 |
| | | New England | 940 | 1005 | 880 | 940 |
| 8″ Rolok-Bak | Cement-Lime | Mississippi | 950 | 905 | 965 | 940 |
| | | New England | 815 | 1155 | 680 | 880 |
| | Cement | Mississippi | 925 | 890 | 950 | 920 |
| | | New England | 1270 | 1245 | 1105 | 1205 |
| 12″ Rolok-Bak (heavy-duty) | Cement-Lime | Mississippi | 775 | 830 | 950 | 850 |
| | | New England | 1150 | 830 | 910 | 965 |
| | Cement | Mississippi | 1035 | 835 | 955 | 940 |
| | | Mississippi | 1025 | 905 | ..... | 965 |
| | | New England | 1635 | 1525 | 1610 | 1590 |
| | | New England | 1400 | 1250 | ..... | 1325 |
| 12″ Rolok-Bak (standard) | Cement | Mississippi | 1010 | ..... | ..... | 1010 |
| | | New England | 1160 | ..... | ..... | 1160 |
| 4″ "Economy" | Cement-Lime | Mississippi | 1370 | 1365 | 1565 | 1435 |
| | | New England | 1960 | 1975 | 1880 | 1940 |
| | Cement | Mississippi | 1650 | 1350 | 1875 | 1625 |
| | | New England | 2755 | 3520 | 3160 | 3145 |

*Tested at 57 to 62 days.

The walls in Table No. 12 were built under contract. Bids were obtained from a number of brick masons and the work awarded to the lowest bidder. With one exception, the mason received neither instructions nor supervision. In this work there was practically no mortar in the longitudinal vertical joints, the horizontal mortar beds were deeply furrowed and the brick were laid at a high rate. There was one interesting exception in the walls built under contract. In one of the 8″ solid walls of Mississippi

brick laid in cement mortar (5th line of table), the mason was instructed to build with completely filled mortar joints. The compressive strength of this wall was 1480 lb. per sq. inch. The same mason built another wall (fourth line) with the same type brick and mortar, but the workmanship was similar to that performed on the other contract walls. The compressive strength of that wall was only 870 lbs. per sq. inch, or approximately 60% of the wall built with good workmanship.

## TABLE NO. 12

## RESULTS OF COMPRESSIVE TESTS ON BRICK WALLS

### Series "B"

### (Uninspected Workmanship)

| Type of Wall | Mortar | Kind of Brick | Wall Strength* lbs./sq. inch | | | |
|---|---|---|---|---|---|---|
| | | | a | b | c | Average |
| 8″ Solid | Lime | Chicago | 255 | 250 | 315 | 275 |
| | Cement-Lime | Chicago | 575 | 620 | 590 | 595 |
| | Cement | Chicago | 660 | 750 | 580 | 665 |
| | Cement | Mississippi | 870 | ..... | ..... | 870 |
| | Cement | Mississippi | 1480 | ..... | ..... | 1480 |
| | Cement | New England | 2030 | ..... | ..... | 2030 |
| 12″ Solid | Lime | Chicago | 295 | 350 | 250 | 300 |
| | Cement-Lime | Chicago | 580 | 575 | 580 | 580 |
| | Cement | Chicago | 655 | 655 | ..... | 655 |
| 8″ All-Rolok | Cement | Mississippi | 440 | ..... | ..... | 440 |
| 8″ All-Rolok in Flemish Bond | Cement | Mississippi | 535 | ..... | ..... | 535 |
| 8″ Rolok-Bak | Cement | Mississippi | 745 | ..... | ..... | 745 |

*Tested at 57 to 62 days.

The average strengths of solid walls, which had completely filled vertical joints and smooth spread horizontal mortar beds, were as follows:

## TABLE NO. 13

## AVERAGE COMPRESSIVE STRENGTH OF WALLS

| Kind of Brick | Shearing strength of brick | Compressive strength of half brick, flat-wise | Average Compressive Strength of Solid Walls | |
|---|---|---|---|---|
| | | | Cement-lime Mortar | Cement Mortar |
| | lbs./sq. in. | lbs./sq. in. | lbs./sq. in. | lbs./sq. in. |
| Chicago | 1100 | 3280 | .... | 895 |
| Detroit | 1165 | 3540 | 945 | 1145 |
| Mississippi | 1590 | 3410 | 1300 | 1550 |
| New England | 3550 | 8600 | 1875 | 2855 |

The following comments have been copied from the conclusion given in Research Paper No. 108:

"The strengths of the solid walls were more closely related to the shearing strength of the brick than to any other strength property measured. The compressive strength of the half brick flatwise appeared to be the next best measure and was better than the compressive strength on edge, the modulus of rupture, or the tensile strength of the brick.

The strength of the solid walls, which were built by contract, varied about as the cube root of the compressive strength of the mortar cylinders (2 inches in diameter and 4 inches long) which were made from the mortar of the walls and cured under the same conditions. For the solid walls, built under careful supervision, the increase in strength for cement-mortar walls over those laid in cement-lime mortar was about 20% for walls of Detroit and Mississippi brick and about 50% for walls of New England brick, while the average ratio of the cube roots of the mortar cylinder strengths (cement and cement-lime mortars) was 1.38.

With brick, the cross sections of which closely approximated rectangles, the strengths of the hollow walls varied about as the net areas in compression. When the brick were warped, the strength of the hollow walls was found to be less than that expected from the net area.

The condition of the horizontal mortar beds in the walls affected the wall strength. Walls in which the beds were smooth were stronger than walls in which the mortar beds were furrowed by from 24 to 109 per cent."

As part of its program of research on Building Materials and Structures, the National Bureau of Standards on Nov. 21, 1938, issued its Report BMS5, "Structural Properties of Six Masonry Wall Construction", by Whittemore, Stang, and Parsons. Three of the wall types were solid brick walls, illustrated in Figure 9.

Two types of brick, two types of mortar, and two types of workmanship were used in an effort to determine the probable range of properties which might be expected in good practice. Tests were made for compressive and transverse strength, concentrated loading, impact and racking. Ordinarily, three specimens of each type of construction were subjected to each of the five tests.

One kind of brick was of shale, formed by the stiff-mud, sidecut process; while the other brick were of clay, formed in sanded molds by the soft-mud process. The following table gives the physical properties of the two types of brick.

**PHYSICAL PROPERTIES OF BRICK**

| Description of brick | Compressive strength lb./sq. in. | Modulus of rupture lb./sq. in. | 24-hr. cold C % | 5-hr. boil B % | Saturation Coefficient C/B ratio | 1-min. partial immersion† | |
|---|---|---|---|---|---|---|---|
| | | | | | | Dry | As laid |
| Stiff-mud, shale. | 17,600 | 2,275 | 1.9 | 3.45 | 0.53 | 8 | 8 |
| Soft-mud, sand-molded..... | 2,670 | 550 | 11.3 | 15.1 | 0.74 | 23 | 11 |

†Immersed on flat side in ⅛ in. of water. Absorption in grams per brick.

The cement mortar was 1 part cement, 0.11 part hydrated lime, and 2.6 parts dry sand, by weight. The proportions by volume were 1 part cement, 0.25 part hydrated lime, and 3 parts loose damp sand, assuming that portland cement weighs 94 lb./cu. ft.,

dry hydrated lime 40 lb./cu. ft., and 80 lb. of dry sand is equivalent to 1 cu. ft. of loose damp sand.

The cement-lime mortar was 1 part cement, 0.42 part hydrated lime, and 5.1 parts dry sand, by weight. The proportions by volume were 1 part cement, 1 part hydrated lime, and 6 parts loose damp sand.

**PHYSICAL PROPERTIES OF THE MORTARS**

| Kind of Mortar | Water content, by weight of dry materials % | Flow % | *Compressive strength lb./sq. in. | |
|---|---|---|---|---|
| | | | Air Storage | Water Storage |
| Cement | 19.6 | 113 | 1390 | 3220 |
| Cement-lime | 23.3 | 107 | 450 | 640 |

*2 inch cubes tested at 28 days.

The walls of type AA were built with the shale brick laid in common American bond and cement mortar. All joints were completely filled with mortar and face joints cut flush.

The walls of types AB and AC were built with the sand-molded clay brick laid in common American bond and cement-lime mortar. The joints in type AB were not completely filled with mortar. The bed joints were furrowed, the collar joints were left open and only the outside of the head joints was filled by lightly buttering the outer edges of the brick. All face joints were cut flush.

The workmanship on walls of type AC was similar to that on type AA, i. e., all joints were completely filled with mortar and face joints cut flush with the surface.

The compressive loads were applied eccentrically along the third point. In theory, a load applied at the third point will produce, at the extreme edge nearer the eccentric load, a unit compressive stress which is double the average unit stress normally obtained by dividing the total load by the cross-sectional area; while the unit stress at the other edge approaches zero.

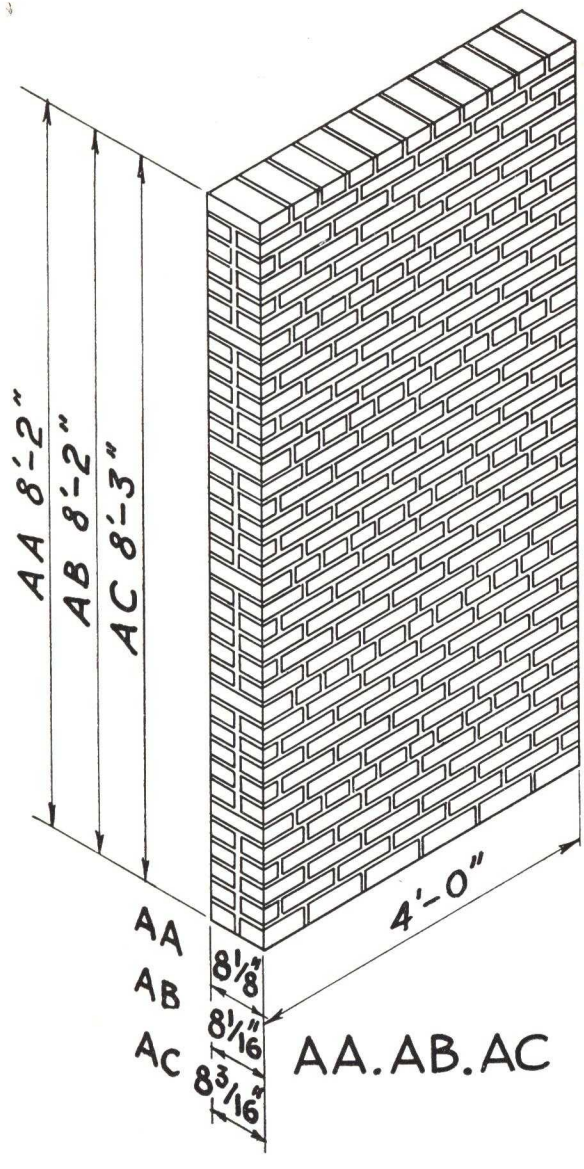
In these tests, however, the exact conditions to satisfy the theory did not exist, principally because the walls were capped by a rigid horizontal steel plate. The compressometer readings taken during the test indicate that the stresses were not distributed in exact agreement with this theory. There was some compressive stress at the far edge, approximately one-eighth the stress at the near edge. It is probable, therefore, that the unit compressive stress at the extreme edge near the eccentric load would be 75 to 85% greater than the average unit compressive stress.

The following table gives the maximum compressive loads on brick masonry walls as reported in BMS5.

| Wall Type | Maximum Compressive Load in kips/lin. ft.* | | | Average in kips/lin. ft. |
|---|---|---|---|---|
| AA | 249 | 378 | 344 | 324 |
| AB | 63.2 | 52.5 | 65.8 | 60.5 |
| AC | 90.5 | 110.0 | 102.5 | 101.0 |

*Tested 28 days.

Load applied at one-third the thickness of the specimen from the inside face.




Courtesy of National Bureau of Standards

*Figure 9*
*Dimensions of three wall types.*

The maximum unit compressive stress at the inside edge has been estimated as follows:

| Wall Type | Ultimate Compressive Stress in lb./sq. in. | | | Average in lb./sq. in. |
|---|---|---|---|---|
| AA | 4650 | 7000 | 6350 | 6000 |
| AB | 1180 | 980 | 1225 | 1130 |
| AC | 1660 | 2020 | 1880 | 1850 |

### 303. RELATION BETWEEN WALL STRENGTH AND STRENGTH OF BRICK

The accompanying chart illustrates the general relation between the compressive strength of brick and the compressive strength of walls built with two kinds of mortar. The chart is based on data from the reports of the National Bureau of Standards (R.P. No. 108) and the curves represent the averages of a number of tests. The compressive strengths of the cement mortar (1-1/10-3 mix) and cement-lime mortar, (1-1-6), tested in 2″ x 4″ cylinders, were 1950 and 750 pounds per square inch respectively. The walls were 12″ thick and all workmanship was inspected. All horizontal beds were smooth (not furrowed) and all vertical joints were completely filled. The panels were tested under concentric loading, uniformly distributed.

The curves indicate the increased wall strengths obtained with brick of higher strength. Wall strengths are increased also, but to a lesser extent, by the use of higher strength mortar. In most structures, the loads imposed upon the brick masonry are generally comparatively low and extremely high strengths are not necessary. Where concentrated loads occur, the required strength seldom approaches the allowable working strength of the brick masonry. If necessary, however, an increase in wall strength may be obtained by increasing the thickness of the wall under the concentrated load (piers or pilasters) or by increasing the mortar strength.

The more recent tests (1938) made by the National Bureau of Standards and summarized in the preceding section indicate that the compressive strength of the brick masonry walls as related to the compressive strength of the brick unit is greater than that shown on the accompanying chart. It is apparent, therefore, that the chart is conservative and may be used as a reliable estimate for approximating the expected compressive strength of brick masonry walls built with reasonably good workmanship in either of the two types of mortar mixes.

### 304. TRANSVERSE STRENGTH

While most walls are designed to resist compressive stress, they are sometimes called upon to resist bending caused by forces normal to the wall face, as in the case of exterior basement walls, storage bin walls, wind pressure on large surfaces or walls in regions subject to earthquakes or concussion from explosives, etc. It is in such instances that transverse strength is important.

In plain brick masonry, the transverse strength may be measured by the strength of the mortar joints in tension or by the strength of adhesion between bricks and mortar. Failure in the mortar joints is more likely to occur than failure of the brick in tension, bending or shear. Maximum transverse strength may be developed by attention to (1) the quality of mortar, (2) workmanship producing smooth mortar beds and full joints and (3) properly wetted brick. Superimposed compressive loads add to the resistance of a wall to transverse stresses. The addition of steel reinforcement greatly increases the lateral strength of a wall and the design of such construction is discussed in chapters on Reinforced Brick Masonry.

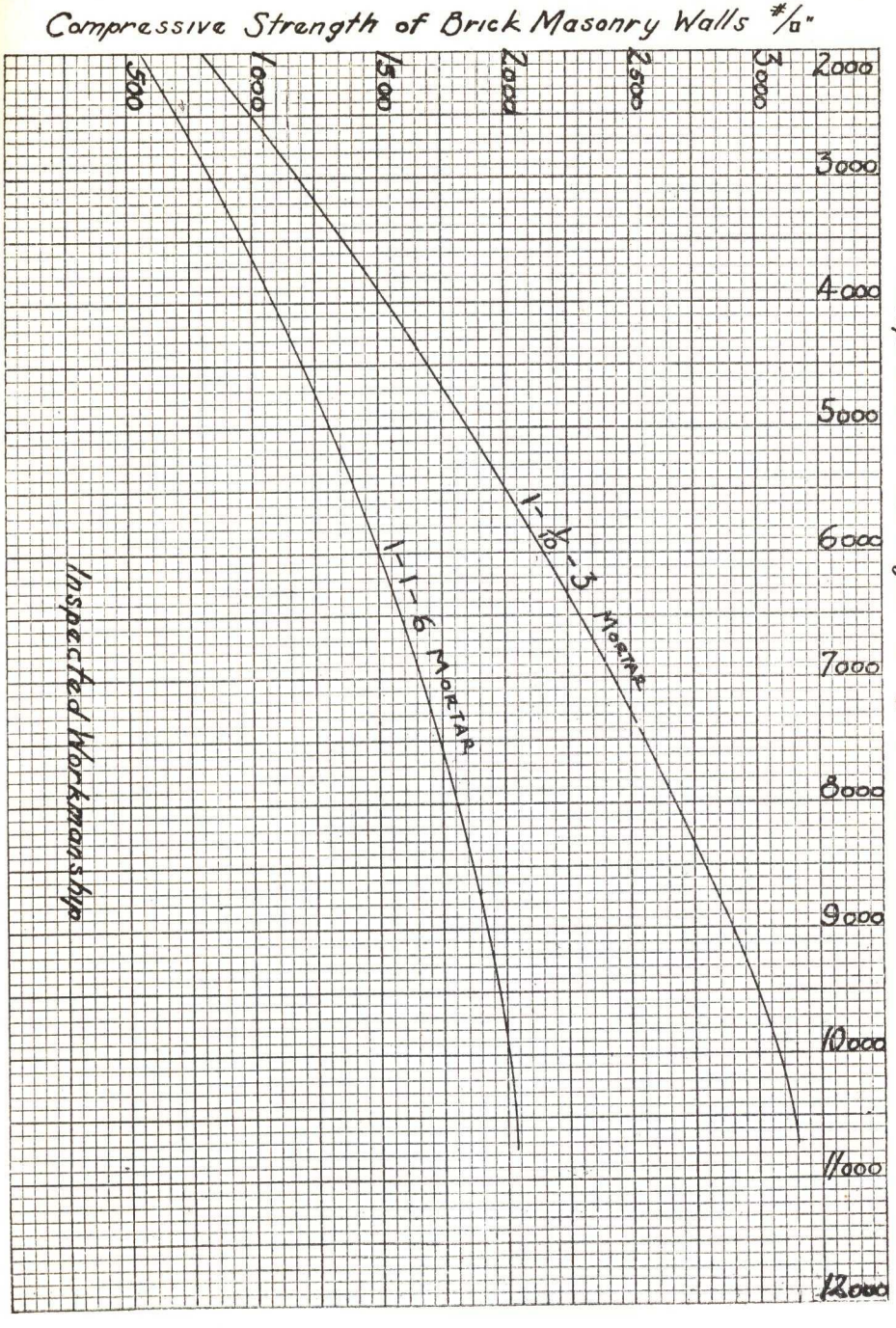



*Figure 10*

*Showing the relation between compressive strength of brick and brick masonry.*

The resistance of a wall to bending due to a lateral force is also expressed in terms of its Modulus of Rupture, which is the unit stress computed from the following formula:

$$S = \frac{Mc}{I}$$

Where, S is unit stress in extreme fiber; Modulus of Rupture;

M is bending moment at Ultimate Load;

c is distance from neutral axis to the extreme fiber;

and I is the moments of inertia of the section.

The Modulus of Rupture of a wall section is determined from the transverse loading necessary to rupture the wall specimen. If the load, W, is expressed in terms of equivalent uniform load per foot width of the wall, the bending moment, M, is $\frac{Wl}{8}$. If the width of the unit section of wall is assumed as b and the thickness of the wall as d; the value of c may be taken as $\frac{d}{2}$, and the moment of inertia of the rectangular section at right angles to the face of the wall is $\frac{bd^3}{12}$. Substituting in the above equation, the Modulus of Rupture becomes,

$$S = \frac{Mc}{I} = \frac{Wl/8xd/2}{bd^3/12} = \frac{3Wl}{4bd^2},$$

in which, W is equivalent uniform load in lb. per foot width of wall

$l$ is vertical distance between supports in inches

b is breadth or width of unit wall section in inches (assumed 12″)

d is depth or thickness of wall section in inches.

As an example, the transverse tests on brick walls made at the National Bureau of Standards in 1938, give an average load of 125 lb./sq. ft. for wall type AA; the equivalent uniform load, W, between the supports, 7′-6″ apart, 125 x 7½ = 937.5 lb.; the distance between supports, l, is 90 inches; b = 12 inches and d = 8.125 inches. The modulus of rupture is solved as $S = \frac{3Wl}{4bd^2} = \frac{3 \times 937.5 \times 90}{4 \times 12 \times 8.125^2} = 80$ lb./sq. in.

The transverse strength of plain brickwork in flexure was studied at the National Bureau of Standards in 1928 and again in 1938. In the 1928 studies, three brick panels, 6 feet wide by 9 feet high, were supported at the top and bottom and subjected to lateral concentrated loads at the third points. The bricks were laid in a 1:3 cement mortar; had a flat compressive strength of 3280 lb. per sq. in. and a modulus of rupture of 1225 lb. per sq. in. The results obtained are shown in Table No. 14.

More recent (1938) tests for transverse strength were made at the National Bureau of Standards. The three types of walls, AA, AB and AC, are described in Section 302. The walls were approximately 4 ft. wide, 8 ft. high and 8 inches thick. Three specimens of each type were tested in a manner similar to that described for the 1928 tests. The walls were supported at the top and bottom with 7′-6″ between supports and the load applied at two quarter points.

The results of tests for transverse strength given in the accompanying tables will include both the ultimate equivalent uniform load in lb. per sq. ft. and the Modulus of Rupture in lb. per sq. in.

## TABLE NO. 14

## TRANSVERSE TESTS OF BRICK WALLS MADE AT THE NATIONAL BUREAU OF STANDARDS — 1928

| Wall No. | Span (Approx.) | Thickness of Wall in Inches | Kind of Mortar | Total Applied Concentrated Load Applied at the Third Point | Equivalent Uniform Load per sq. ft. | Modulus of Rupture lbs. per sq. in. |
|---|---|---|---|---|---|---|
| 1 | 9′-0″ | 8 | 1:3 Cement | 4397 lbs. | 109 | 96 |
| 2 | 9′-0″ | 8 | 1:3 Cement | 2380 lbs. | 59 | 59 |
| 3 | 9′-0″ | 12 | 1:3 Cement | 8925 lbs. | 222 | 82 |

The results of the 1938 tests were published in the National Bureau of Standards Report BMS5 and the following table has been compiled from the data therein.

## TRANSVERSE TESTS ON BRICK WALLS MADE AT NATIONAL BUREAU OF STANDARDS — 1938

| Wall Type | Equivalent Uniform Load (lb./sq. ft.) | | | | Modulus of Rupture (lb./sq. in.) | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | Average | 1 | 2 | 3 | Average |
| AA | 115 | 120 | 140 | 125 | 73.6 | 76.7 | 89.5 | 79.9 |
| AB | 53.3 | 38.0 | 52.3 | 48 | 34.7 | 24.7 | 34.0 | 31.1 |
| AC | 85 | 80 | 82 | 82 | 53.6 | 50.4 | 51.7 | 51.9 |

Tested at age of 28 days.

Tests were made several years ago at the University of California by Raymond E. Davis, Professor of Civil Engineering, for the purpose of determining the transverse strength of brick masonry composed of various kinds of brick laid in two types of mortar.

Flexure tests were made on brick wall sections tested as beams at the ages of 2 months, 6 months, and 12 months. In order to simulate the increasing load of additional brickwork upon the lower portions of a brick building during construction, one group of specimens was subjected to an increasing dead load each day for the first 20 days after being made, the total load at the end of 20 days remaining until just before testing. The modulus of rupture of specimens thus loaded was about 40% greater than that of similar specimens not thus loaded.

## TABLE NO. 15

## MODULUS OF RUPTURE OF BRICK MASONRY

| Compressive Strength of Brick lb./sq. in. | 1C-1L-6S Mortar | | | 1C-½L-4½S Mortar | | |
|---|---|---|---|---|---|---|
| | Modulus of Rupture lb./sq. in. | Ultimate transverse uniform load in lb./sq. ft. 11′ height | | Modulus of Rupture lb./sq. in. | Ultimate transverse uniform load in lb./sq. ft. 11′ height | |
| | | 8″ wall | 12″ wall | | 8″ wall | 12″ wall |
| 3446 | 96 | 65 | 150 | 178 | 125 | 280 |
| 4280 | 110 | 75 | 170 | 261 | 185 | 410 |
| 7714 | 201 | 140 | 320 | 408 | 285 | 650 |
| 3446* | 156 | 110 | 250 | | | |

* Loaded while curing.

Table No. 15 shows the Modulus of Rupture of brick masonry obtained by tests for various brick and mortars (each value representing an average of a group of specimens); and also the computed ultimate uniform transverse load for 8-inch and 12-inch brick walls of indefinite length having a clear height of 11 feet (a usual clear height maximum per story), using the test values as a basis.

## 305. TENSILE STRENGTH OF BRICKWORK

It is recognized that the tensile strength of the individual brick and of the brickwork may play an important part in reinforced brick masonry beam construction, particularly in resisting diagonal tension. In ordinary brickwork, however, tensile strength is generally governed by the limiting value of the bond that exists between the brick and mortar and points to the necessity of developing the maximum value for such bond by means of good workmanship and the use of a mortar having a combination of strength and plasticity and brick which has been properly wetted. Mechanical bond by means of perforated or scored bricks is an additional aid.

In connection with "Shear Tests of Reinforced Brick Masonry Beams", by Parsons, Stang and McBurney, National Bureau of Standards, R. P. No. 504, tests were made of the strength of bond in tension of brick-mortar joints. The specimens consisted of two brick laid flatwise, separated by mortar, the cross section of the joint being 30 square inches for standard size brick. Twenty test specimens were made for each of the following conditions: Two makes of brick (Chicago and Philadelphia), dry brick and dry cure, dry brick and damp cure, wet brick and dry cure, and wet brick and damp cure.

Bricks were considered as dry after one week's storage in a steam heated room. Some brick were used after 48 hours in a drying oven at 220° F., followed by 24 hours' cooling in the laboratory.

For wetting brick, the procedure was to totally immerse previously dried brick for one hour, stand on end in air for one-half hour, and then make the test specimens within the next half hour.

The properties of the brick used in these tests are given in the following table.

**PROPERTIES OF BRICK**

| Kind of Brick | Compressive Strength lb./sq. in. | | Modulus of Rupture | Per cent of Absorption | | |
|---|---|---|---|---|---|---|
| | Flatwise | Edgewise | lb./sq. in. | 5 hr. cold | 48 hr. cold | 5 hr. boil |
| Chicago..... | 3910 | 4280 | 1530 | 8.8 | 10.8 | 14.7 |
| Philadelphia..... | 4510 | 5240 | 650 | 11.1 | 12.7 | 16.1 |

The mortar was proportioned by weight to give a mixture approximately equivalent by volume to 1 part of Portland cement to 3 parts of loose, damp sand, with an addition of hydrated lime equal to 15 per cent of the volume of cement. The weights used were 94 pounds of cement and 6 pounds of hydrated lime to 220 pounds of dry sand, water being added in the amounts desired by the masons.

The mortar was proportioned and mixed dry. Time was counted from the moment of adding the water to the mortar mix. A flow test was made before starting construction of the test specimens. An attempt was made to have the flow immediately after mixing between 110 and 120 per cent. The properties of the mortar are given in the following Table No. 16.

TABLE NO. 16

## PROPERTIES OF MORTAR FOR THE TENSILE AND SHEAR TEST SPECIMENS

**Each value was derived from results of from 40 to 51 tests. Strength tests at ages ranging from 28 to 60 days**

| Property | Mean Value |
|---|---|
| Initial flow.....Per cent | 116 |
| Flow one-half hour after mixing.....Per cent | 92 |
| Tensile strengths of briquettes with: | |
| Dry storage.....lb./sq. in. | 290 |
| Damp storage.....lb./sq. in. | 470 |
| Compressive strengths of 2 by 4 inch cylinders with: | |
| Dry storage.....lb./sq. in. | 3,410 |
| Damp storage.....lb./sq. in. | 4,010 |

The results of the tensile tests are recorded in the following table:

TABLE NO. 17

## RESULTS OF TENSILE TESTS OF BOND BETWEEN MORTAR AND BRICK

**Age at test 28 to 60 days**

| Number of Specimens | Kind of Brick | Condition when laid | Storage | Average Strength lb./sq. in. |
|---|---|---|---|---|
| 16 | Chicago | Dry | Dry | 38 |
| 16 | | | Damp | 38 |
| 20 | | Wet | Dry | 55 |
| 20 | | | Damp | 61 |
| 16 | Philadelphia | Dry | Dry | 18 |
| 17 | | | Damp | 27 |
| 20 | | Wet | Dry | 53 |
| 20 | | | Damp | 44 |

Where test specimens represented wetted brick, the characteristic failure was not at the junction of brick and mortar but was a failure in the brick. Chicago brick left a "skin" adhering to the mortar, and Philadelphia brick frequently pulled off or sheared off their flats to a depth of one-eighth of an inch. In other words, when brick had been wetted, bond between brick and mortar exceeded the strength of the brick. On the other hand, a few of the "dry brick" specimens showed separation in the mortar, mortar adhering to both brick. The difference between number of tests indicated and the 20 test specimens originally constructed represents failures of bond occurring by handling the specimens. The number of tests is too few and the variation too great to permit much weight to be given either to averages or distributions, but it is evident that wetting Chicago and Philadelphia brick greatly increased the strength of the bond.

## 306. SHEARING STRENGTH

When forces act upon a body in parallel lines or planes, shearing stresses are produced in that portion of the body between the planes of the forces. The average unit shearing stress is calculated by dividing the resultant force in any plane by the area of the section along that plane.

The shearing stress of masonry is an important factor in the resistance of masonry walls to lateral forces experienced in skeleton frame structures. Shearing stresses may also be produced in a wall by the action of wind or other forces on adjoining perpendicular walls. As in tensile strength, the shearing strength of the mortar joint is probably the governing value in most cases.

In his test on masonry assemblies, Professor R. E. Davis, mentioned before, found that the direct shearing strength of his specimens of brick masonry was approximately 86 lb. per sq. inch for brick masonry laid in 1C-1L-6S mortar and 130 lb. per sq. inch when built with 1C-½L-4½S mortar.

Research Paper No. 504, referred to in Section 305, reports tests made for shearing strength of brick-mortar specimens. These shearing specimens were made with similar brick and the same mortar mixture and procedure as the brick-mortar tensile specimens. The specimens were made by laying three brick flat, the top and bottom brick having their ends in line while the center brick was displaced lengthwise from one-half to three-quarters of an inch.

The projecting ends of the two outer bricks of each specimen were capped with plaster of Paris, the surfaces of both caps being in one plane. The projecting end of the center brick was also capped, its surface being, as far as possible, parallel to the surfaces of the two other caps.

The specimens after capping were loaded in compression, the load being applied to the center brick and the specimen resting on the ends of the two outside brick.

Results of tests on brick-mortar shearing specimens are given in the following table:

**RESULTS OF SHEAR TESTS OF BOND BETWEEN MORTAR AND BRICK**
**Age at test, 28 to 60 days**

| Number of Specimens | Kind of Brick | Condition when laid | Storage | Average Strength lb./sq. in. |
|---|---|---|---|---|
| 16 | Chicago | Dry | Dry | 100 |
| 12 | | | Damp | 115 |
| 20 | | Wet | Dry | 275 |
| 19 | | | Damp | 245 |
| 10 | Philadelphia | Dry | Dry | 91 |
| 11 | | | Damp | 120 |
| 19 | | Wet | Dry | 231 |
| 20 | | | Damp | 173 |

In general, results of these tests are similar to the results of tests on the brick-mortar tensile specimens. The average shearing strength of the mortar and brick laid dry was 106 lbs. per sq. inch, while the average shearing strength when the brick were laid wet was 231 lbs. per sq. in. It may be noted also that specimens made with dry brick became stronger under damp curing conditions, while those made with wet brick were stronger when dry cured in the air.

In the tests of reinforced brick masonry beams, reported in Research Paper 504, the maximum shearing stress in the brick masonry was determined. The average maximum shearing stress in the beams built with all brick laid flatwise in running bond and staggered joints was 159 pounds per sq. in. for the Chicago brick and 97 lbs. per square inch for the Philadelphia brick. More complete data regarding the results of the RBM beam test is given in Chapter 9.

Professor M. O. Withey, in a series of tests on brick masonry beams at the University of Wisconsin (see A.S.T.M. Proc. Vol. 33, Part II), reported shearing stress from 91 to 294 lbs. per sq. in. Three series of beams were tested, each series having a different make of brick. There were twenty-five beams in all. The mortar in all beams was 1 part portland cement, 1/3 part hydrated lime and 4 parts sand by weight.

In the aforementioned National Bureau of Standards Report BMS5 (1938), results are given for the racking tests on wall types AA, AB and AC, described in Section 303. In this test, the bottom edge of wall specimens, approximately 8 feet long and 8 feet high, is made secure to a rigid horizontal support and a load parallel to the support applied to the top of the specimen at one end. The racking test is intended to simulate the forces on the end of a wall which produce shearing stresses.

The shearing strength of all specimens was sufficient to resist the load of 50,000 lb., approximately 65 lb. per sq. in., which was the maximum thrusting capacity of the apparatus. There was no sign of a crack or rupture in any of the specimens. Specimens of wall type AA had a deformation in the eight foot height of less than 0.04 in., with no measurable set upon release of the load. The deformation in the eight foot height of specimens of wall type AB at maximum load averaged .07 in., with a permanent set of less than .03 inch. Wall type AC had an average deformation at maximum load of less than .03 inch, with an average permanent set of .005 inch.

## 307. WEATHER RESISTANCE

The resistance of a masonry wall to weathering is dependent upon the design of the wall, quality of materials and workmanship. The design is of primary importance and should provide: (1) Adequate flashing, (2) Properly tooled joints, (3) Tightly caulked door and window frames, (4) sufficient wash or slope to readily drain all horizontal and projecting surfaces, sills, etc., including the overhang and drip, (5) Adequate gutters and downspouts. A design which helps eliminate the possibility of moisture or water entering the wall will contribute greatly to the weather resistance and all-around satisfactory performance of the brick masonry.

Brick masonry is practically immune to disintegration from chemical or biological attack, or movements caused by temperature or moisture changes. Alternate freezing and thawing has very little effect on well-burned brick, but will, under severe conditions, attack most under-burned brick.

Severe weathering conditions are the result of two factors: climate and exposure of wall. Freezing and thawing under relatively dry conditions has little effect on any building material. Freezing is most severe when it occurs shortly after a rain and the wall surface is saturated with water. The climate in the northeastern part of the United States is typical of the conditions which are most severe on building materials.

In this geographical belt of severe weathering, the wall or walls exposed to the prevailing winds which bring the rain and the cold weather will be subjected to the worst conditions. In many buildings, the heat within keeps the walls sufficiently warm to discourage freezing at all, or at least, to delay it until the walls have had an opportunity to dry considerably, if not completely. There are, however, two locations in the wall of a building which do not enjoy the benefit of this heat from within: the parapet and foundation. This is also true in retaining walls, garden walls, bridge abutments, porch floors, terraces, steps, walls, driveways, etc.

In such locations only well-burned brick should be used, such as Grade SW of the A.S.T.M. Tentative Specification For Building Brick, Designation: C 62-37T. For walls of buildings above the foundation wall and below the parapet wall, Grade MW brick should give satisfactory service. Brick of Grade NW may be used for backup of all building walls, excepting the parapet which should be composed entirely of Grade SW brick.

The mortar used in masonry walls, subject to severe weathering, should be as recommended in Chapter 2.

Good workmanship is essential to weather resistant masonry. All joints should be completely filled with mortar and all exposed joints tooled so as to produce either the weathered or concave joint.

## 308. MOISTURE PENETRATION

Moisture penetrates brick masonry walls principally through openings between the mortar and the brick, rather than through the masonry units or the mortar, although a highly absorbent brick used as a through header may transmit sufficient moisture capillarity to cause objectionable dampness if plaster is applied directly to the back of the wall.

In view of the practical difficulty of eliminating all openings between brick and mortar, cavity walls or hollow walls without through masonry headers are recommended as preferred types of construction. These walls provide a positive barrier in the form of an air space, to the transmission of moisture from the outer to the inner wythe of masonry, and provision is made to discharge from the cavity any moisture which may enter through the outer wythe.

It is also recommended that all solid masonry walls containing through headers be furred before plastering, not only as a guarantee against any possible moisture penetration, but also to decrease the coefficient of heat conductivity of the wall and thereby reducing the possibility of moisture condensation.

The subjects of moisture penetration and efflorescence including recommendations for flashing, are discussed in Chapter 8.

## 309. FIRE RESISTANCE

Brick masonry is more fire resistive than most any other form of construction and has a high salvage value after exposure to fire.

Research and testing have developed accurate information regarding maximum temperatures and duration of severe fires. It has been found that maximum temperatures rarely exceed 1700° F. The duration, of course, depends upon the size and construction of the structure and more especially upon the nature and amount of the contents. Structures classified by Building Codes as "Fireproof" are required to resist exposure to fires of 4 hours duration.

The A.S.T.M. Standard Specifications for Fire Tests of Building Construction and Materials, C19-33, have been approved under the procedure of the American Standards Assn. as an American standard and tests conducted in accordance with these specifications are used almost universally in the United States as a basis for establishing fire resistive ratings of various types of construction. These specifications cover a fire endurance test and a hose stream test.

Briefly described, the fire endurance test consists in exposing the specimen to controlled temperatures on one side, in accordance with a time temperature curve. Points on the curve that determine its character are:

| | |
|---|---|
| 1000° F. . . . . at 5 minutes | 1850° F. . . . . at 2 hours |
| 1300° F. . . . . at 10 minutes | 2000° F. . . . . at 4 hours |
| 1550° F. . . . . at 30 minutes | 2300° F. . . . . at 8 hours or over |
| 1700° F. . . . . at 1 hour | |

Wall and partition specimens must have an area of not less than 100 sq. ft. and floor and roof specimens an area of not less than 180 sq. ft. The specification provides "The tests shall not be regarded as successful unless the following conditions are met:

(a) The wall or partition shall have withstood the fire endurance test without passage of flame or gases hot enough to ignite cotton waste, for a period equal to that for which classification is desired.

(b) The wall or partition shall have withstood the fire and hose stream test without passage of flame, or gases hot enough to ignite cotton waste, or of the hose stream.

(c) The fire-stopping, if any, shall have functioned to prevent passage of fire for a period equal to that for which classification is desired.

(d) Transmission of heat through the wall or partition during the fire endurance test shall not have been such as to raise the temperature on its unexposed surface more than 250° F. (139°C) above its initial temperature."

For bearing walls there is an additional requirement that during the fire endurance and hose stream tests the construction shall be loaded "in a manner calculated to develop theoretically as nearly as practicable the working stresses contemplated by the designer."

The hose stream test consists in exposing the specimen to the standard fire exposure for a period equal to one-half of that indicated as the resistance period but not for more than 1 hour, immediately after which the specimen is subjected to the impact, erosion, and cooling effects of a hose stream, delivered through a standard fire nozzle at pressures varying from 30 to 45 lbs./sq. in., depending upon the resistance period. Constructions are classified according to their ability to withstand the standard tests, but the tests are not a measure of the suitability of the construction for use after fire exposure.

Fire ratings of brick masonry have been determined through an extensive series of tests at the Bureau of Standards, described in Bureaus of Standards Letter Circular No. 228, as follows:

| Classifications | Rating |
|---|---|
| 4 in. Interior partitions. Unplastered | 1 hour |
| 4 in. Interior partitions. Plastered both sides | 2 hours |
| 8 in. Interior or exterior walls. Non-bearing with incombustible structural members, framed in | 5 hours |
| Bearing, with combustible structural members, framed in | 2 hours |
| 12 in. Interior or exterior walls. Non-bearing or bearing. Not less than | 9 hours |

Gypsum plaster ¾" thick on either side of the wall will usually increase the classifications at least one-half hour.

## 310. HEAT TRANSMISSION COEFFICIENTS

The total heat losses from any building may be classified as either transmission or infiltration losses. This fact is stated in the 1936 Guide of the American Society of Heating and Ventilating Engineers as follows:

> "The sum of the heat losses by transmission through the outside wall and glass as well as through any cold floors, ceiling, or roof, plus the heat equivalent of the cold air entering by infiltration represents the total heat loss equivalent for any building."

Transmission losses only will be considered in this section. Transmission losses represent the heat transmitted through the walls of a building by radiation, conduction,

and convection. The amount of heat transmitted through a wall for any given inside and outside temperatures will depend upon the construction of the wall and the material of which it is constructed. Generally speaking, walls which contain air spaces have lower transmission coefficients than solid walls. The thermal transmittance or over-all coefficient of heat transmission, frequently designated by "U", is the amount of heat expressed in British thermal units transmitted in one hour per square foot of the wall for a difference in temperature of one degree F. between the air on the inside and outside of the wall. The resistance of a wall, designated by "R", is the reciprocal of the transmission and may be defined as the number of hours required for the passage of one Btu of heat through one square foot of wall area for one degree F. temperature difference between the air on one side of the wall and the air on the other. Walls having the lower values of "U" and correspondingly higher values of "R" have the better insulating qualities.

If a wall is made up of several materials, the total resistance of the composite wall is equal to the sum of the resistances of its separate parts including the resistances of air spaces and the surface resistances. Since it would be next to impossible to determine by actual test the resistance to heat transmission of all types of walls used in building construction, it is common practice to calculate the resistance of building walls by adding the resistances of the wall components. The Heating and Ventilating Engineers Guide contains tables of conductivities and conductances for practically all building materials used commercially. These coefficients have been determined by test and are used in computing the resistance of a building wall.

The coefficients of conductance for many building materials vary widely for different grades of the same materials and the coefficients listed in the Guide are, therefore, averages which in some instances amount to little more than approximations. In the case of brick, for example, low density brick is listed as having a conductance of 5.00, while the conductance listed for high density brick is 9.20. The over-all coefficients of heat transmission of the brick walls listed in the ASHVE Guide are based on the conductance of high density brick for the outer 4" of brick masonry and on the conductance coefficient of low density brick for the backup brick. Actually, many brick having the same physical characteristics are used interchangeably in the face of a wall or as backup material. Obviously, the use to which the brick is put has no relation to its conductivity.

Neither the American Society of Heating and Ventilating Engineers, nor the Bureau of Standards, to both of whom inquiries have been addressed, have been able to furnish data which would make it possible to correlate these two values of conductivity with other physical properties of the brick.

An error which frequently creeps into semi-technical discussions is the tendency of some writers to place undue importance upon the result of accurate calculations regardless of the reliability of the data upon which the calculations are based. It requires only a moment's consideration to realize that if the fundamental data be from 15 to 20 per cent inaccurate, the final results will be of the same order of accuracy regardless of the precision of the intermediate calculations.

The difference between the coefficients of conductance of low density brick and high density brick masonry, as listed in the Guide, is slightly over 80% and yet there is no data available which would enable us to specify the type of brick to which either coefficient should be applied.

Similar variations undoubtedly exist in other materials, possibly to a greater or lesser extent. In applying the coefficients listed in the Guide, therefore, they should not be considered as absolutely accurate for all materials and too much importance should

not be attached to relatively slight variations in the computed resistance of wall assemblages.

The resistance and transmission coefficients of some of the more common types of wall construction are tabulated in the following table:

## TABLE NO. 18

## RESISTANCE (R) AND TRANSMITTANCE (U) OF VARIOUS TYPES OF WALL CONSTRUCTION, COMPUTED FROM COEFFICIENTS RECOMMENDED IN A.S.H.V.E. GUIDE, 1939 EDITION

**Wind Velocity outside assumed at 15 mph.**

| Wall Construction | "R" | "U" |
|---|---|---|
| 8″ low density brick, ½″ plaster on wall | 2.52 | 0.40 |
| 12″ low density brick, ½″ plaster on wall | 3.32 | 0.30 |
| 8″ hollow tile, ½″ plaster on wall | 2.59 | 0.39 |
| 12″ hollow tile, ½″ plaster on wall | 3.42 | 0.29 |
| 4″ high density brick, 4″ low density brick, ½″ plaster on wall | 2.16 | 0.46 |
| 4″ high density brick, 8″ low density brick, ½″ plaster on wall | 2.96 | 0.34 |
| 8″ low density brick, furring, wood lath, and plaster | 3.67 | 0.27 |
| 12″ low density brick, furring, wood lath, and plaster | 4.47 | 0.22 |
| 8″ hollow tile, furring, wood lath and plaster | 3.74 | 0.27 |
| 12″ hollow tile, furring, wood lath and plaster | 4.57 | 0.22 |
| 4″ high density brick, 4″ low density brick, furring, wood lath and plaster | 3.31 | 0.30 |
| 4″ high density brick, 8″ low density brick, furring, wood lath and plaster | 4.11 | 0.24 |
| Stucco, 8″ hollow tile, furring, wood lath and plaster | 3.74 | 0.27 |
| Stucco, 12″ hollow tile, furring, wood lath and plaster | 4.57 | 0.22 |
| 4″ low density brick, 4″ hollow tile, furring, wood lath and plaster | 3.87 | 0.26 |
| 4″ high density brick, 4″ hollow tile, furring, wood lath and plaster | 3.51 | 0.28 |
| 4″ low density brick, 8″ hollow tile, ½″ plaster on wall | 3.39 | 0.29 |
| 4″ low density brick, 8″ hollow tile, furring, wood lath and plaster | 4.54 | 0.22 |
| 4″ high density brick, 8″ hollow tile, furring, wood lath and plaster | 4.19 | 0.24 |
| 10″ cavity wall (4″ high density brick, 2″ air space, 4″ low density brick) ½″ plaster direct on wall | 3.06 | 0.33 |
| 10″ cavity wall (4″ high density brick, 2″ air space, 4″ hollow tile) ½″ plaster direct on wall | 3.26 | 0.31 |
| 4″ high density brick veneer on frame construction | 3.73 | 0.27 |
| 4″ low density brick veneer on frame construction | 4.09 | 0.24 |
| Stucco on frame construction | 3.37 | 0.30 |
| Frame construction (siding, sheeting, studding, wood lath and plaster) | 4.07 | 0.25 |

The thermal resistance and conductance coefficients listed in the preceding table are based upon the conductances or resistances of various elements of building construction listed in Table No. 19.

## TABLE NO. 19

## TABLE OF HEAT TRANSMISSION COEFFICIENTS

### from A.S.H.V.E. Guide (1939)

| Material | Transmittance (U) or Conductance (C) | Resistance (R) $\left(R = \frac{1}{U} = \frac{1}{C}\right)$ |
|---|---|---|
| Inside Surface (Still air) | ($f_i$) 1.65 | ($1/f_i$) 0.61 |
| Outside Surface Ordinary (15 mph. wind velocity) | ($f_o$) 6.00 | ($1/f_o$) 0.17 |
| 4″ Brick (low density) | 1.25 | 0.80 |
| 8″ Brick (low density) | 0.62 | 1.60 |
| 12″ Brick (low density) | 0.42 | 2.40 |
| 4″ Brick (high density) | 2.30 | 0.44 |
| 8″ Brick (high density) | 1.15 | 0.88 |
| 12″ Brick (high density) | 0.76 | 1.32 |
| 4″ Hollow Tile | 1.00 | 1.00 |
| 6″ Hollow Tile | 0.64 | 1.57 |
| 8″ Hollow Tile (2 cells in direction of heat flow) | 0.60 | 1.67 |
| 10″ Hollow Tile | 0.58 | 1.72 |
| 12″ Hollow Tile (3 cells in direction of heat flow) | 0.40 | 2.50 |
| 16″ Hollow Tile (1-10″ & 1-6″, ea. having 2 cells in direction of heat flow) | 0.31 | 3.23 |
| Stucco (1″ thick) | 12.00 | 0.08 |
| Cement Mortar (½″ thick) | 24.00 | 0.04 |
| Air Space (more than ¾″) 40° F. mean temp | (a) 1.10 | (1/a) 0.90 |
| Wood lath and plaster (total thickness, ¾″) | 2.50 | 0.40 |
| Metal lath and plaster (total thickness, ¾″) | 4.40 | 0.23 |
| Plaster, cement (1″ thick) | 8.00 | 0.13 |
| Plaster, gypsum (1″ thick) | 3.30 | 0.30 |
| Frame, 1″ yellow pine or fir sheathing and bld. paper | 0.82 | 1.22 |
| Frame, 1″ yellow pine lap siding | 1.28 | 0.78 |
| Frame, 1″ fir sheathing, bldg. paper and yellow pine lap siding | 0.50 | 2.00 |

To obtain the resistance of a composite wall, the resistances of the various components of the wall are added. As previously stated, the resistance is the reciprocal of the transmittance.

For example, if it is desired to obtain the resistance of a wall consisting of 4″ high density brick, 8″ tile, furring, lath and plaster —
Resistance = 0.17 + 0.44 + 1.67 + 80.90 + 0.40 + 0.61 = 4.19

$$\text{The transmittance, } U = \frac{1}{4.19} = 0.24$$

In the above equation, the first term is the outside surface resistance; the second term is the resistance of 4″ of high density brick; the third term is the resistance of 8″ of hollow tile; the fourth term is the resistance of the air space; the fifth term is the resistance of the lath and plaster; and the sixth term is the inside surface resistance. The total is the thermal resistance (R) of one square foot of wall surface per degree difference in outside and inside temperature. The reciprocal of the resistance is the thermal

transmittance (U) in Btu transmitted in one hour through one square foot of wall per degree difference in outside and inside temperature.

## 311. SOUND TRANSMISSION

Architectural acoustics may include both the correction of sound within the room wherein the sound originates as well as the prevention of transmission of sound through floors and walls of buildings. It is general practice, however, to limit the term "acoustics" to the science of sound within a room, while "sound transmission" considers the transfer or passage of sound of external origin into a room. In this discussion, we are concerned with acoustical sound transmission, rather than electrical or mechanical transmission of sound.

Sound waves striking a wall or partition may be divided into three groups, namely:

a. That part reflected back into the room.

b. The part absorbed by the wall.

c. The remainder which is transmitted through the wall.

In acoustics, all sound that is not reflected is frequently termed as being absorbed. In other words, (b) and (c) might be classed under absorption when considering the acoustical correction of a surface; but in sound transmission, the term, absorption, is applied more correctly to mean that percentage of the sound which is actually absorbed in the wall.

Sound transmission through a wall may be accomplished in three ways. First, the sound waves may travel through air passages, such as cracks in the walls or around doors and windows. Second, the sound waves may be communicated to the small particles of material in the wall and in turn to the air on the other side. Very little sound, however, is transmitted in this manner. The third means of transmission is a bodily vibration of the entire wall or parts of it, caused by the compressions and rarefactions of the sound waves. The vibrations of the wall or partition set up sound waves on the further side.

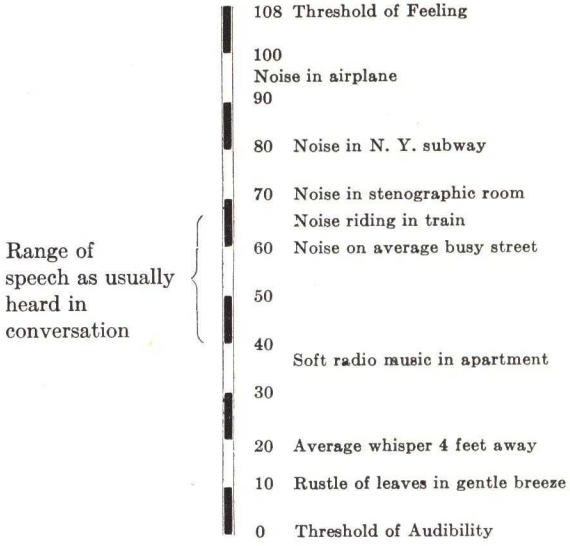
To effectively dampen or stop sound transmission through a wall, there should be no cracks or openings which permit the passage of air through the wall; while the materials comprising the wall and the manner of construction should be such as to discourage the transfer of sound vibrations in the wall as a whole or any segment.

Laboratory experiments have been conducted with various types of partitions to determine their relative effectiveness in reducing or resisting the transmission of sound. Tabulated herein are some of the results of the U. S. Bureau of Standards, the Riverbank Laboratories and the University of Washington.

The resistance to the transmission of sound is usually given as a reduction factor expressed in decibels. The actual physical intensity of sound energy as measured by telephone instruments increases tremendously as compared to the effect recorded by the human ear. Nature protects this organ against excessive stimulation. Experiments show that the response of the ear is proportional to the logarithm of the physical intensity of the sound energy; for instance, sound energies having actual physical intensities of 10, 100 and 1000 would produce effects on the ear in the ratio of 1, 2 and 3 respectively.

The smallest change of power level of sound (sometimes called, "sensation unit") that the ear can detect is approximately one decibel. This is a logarithmic unit (to base 10) and has been adopted by telephone and acoustical engineers to represent different levels of intensity. The range of levels from a soft whisper to very loud speech is approximately 60 decibels. The U. S. Bureau of Standards has published the following scale to illustrate the values of sensation units corresponding to familiar sounds:

## SENSATION UNITS (DECIBELS)




## EAR SENSATION SCALE

The reduction factor given in the following tables is the average of the determinations made at several frequencies. These frequencies range from 250 or less, to more than 3,000.

When comparing the reduction factors of various panels, the relative values should be expressed as a *difference* of their reduction factors in sensation units, and *not* as a *ratio*. For instance, if Panel "A" has a reduction factor of 40 sensation units and that of Panel "B" is 50, Panel "B" will reduce sound heard through it 10 sensation units more than Panel "A." Panel "B" should not be described as 25 per cent better than Panel "A." To illustrate, assume the external sound to have an intensity of 60 sensation units. The intensity of the sound heard through Panel "A" would be 20 sensation units, and that through Panel "B" would be 10. Panel "B" in this case, would be twice as good as Panel "A." On the other hand, assume the original intensity to be 90 sensation units. As heard through Panel "A," the sound would have an intensity of 50, and through Panel "B" an intensity of 40. Under this condition, Panel "B" would be only 20 per cent better than Panel "A." The difference in the sound heard through the two panels remains always as 10 sensation units.

**PAUL E. SABINE, Ph.D.**
**RIVERBANK LABORATORIES, GENEVA, ILLINOIS**

| Material in Test Panel | Wt. per Sq. Ft. Lb. | Average Reduction Factors |
|---|---|---|
| 3″ Solid Gypsum Tile — with 1¼″ Gypsum plaster | 25.4 | 34.3 |
| 3½″ Gypsum Plaster — on metal lath | 32.5 | 36.1 |
| 2 x 4″ frame — wood lath and plaster | 18.0 | 29.4 |
| 4″ Haydite — 1″ Gypsum plaster | 23.2 | 38.0 |
| 8″ Haydite — 1″ Gypsum plaster | 43.5 | 40.1 |

## U. S. BUREAU OF STANDARDS TESTS

| Material in Test Panel | Wt. per Sq. Ft. Lb. | Average Reduction Factors |
|---|---|---|
| 12″ Hollow Tiles — two different types of units, plaster both sides, brown and white finish | 65 | 48.6 |
| 12″ Hollow Tiles — two different types of units, plaster both sides, brown and white finish | 66 | 50. |
| 8″ Hollow Tile — plaster both sides, brown coat and smooth white finish | 48 | 49.8 |
| 6″ Hollow Tile — plaster both sides, brown coat and smooth white finish | 39 | 47.1 |
| 6″ Hollow Tile — plaster both sides, brown coat and smooth white finish | 37 | 45.7 |
| 4″ Hollow Tile — plaster both sides, brown coat and smooth white finish | 29 | 44.0 |
| 4″ Hollow Tile — plaster both sides, brown coat and smooth white finish | 29 | 43.5 |
| 3″ Hollow Tile — plaster both sides, brown coat and smooth white finish | 28 | 44.4 |
| 8″ Hollow Tile (Heath cubes) — plaster both sides, brown coat and smooth white finish | 55 | 51.0 |
| 4″ Hollow Tile — wood furring, paper, metal lath and plaster both sides | 34 | 57.5 |
| 4″ Hollow Tile (on pads) — wood furring, paper, metal lath and plaster both sides | 34 | 58.3 |
| 8″ Hollow Tile — 2 units, 1¾″ apart, filled with Flax-li-num and ½″ Flax-li-num pads at top, bottom and sides of one wythe | 50 | 59.2 |
| 8″ Brick — plaster both sides, brown coat and smooth white finish | 97 | 56.7 |
| 8″ Brick — plaster both sides, brown coat and smooth white finish | 87 | 57.2 |
| 3¾″ Brick — lime plaster with smooth lime finish on both sides — no furring | 49.0 | 50.2 |
| 3¾″ Brick — gypsum plaster with smooth lime finish on both sides — no furring | 49.0 | 53.7 |
| 2¼″ Brick — plaster both sides on wired furring strips | 36.5 | 53.5 |
| 2¼″ Brick — nailed furring strips and plaster both sides | 38.2 | 55.2 |
| 2¼″ Brick — nailed furring strips and Insulite as plaster base | 33.3 | 54.6 |
| 2¼″ Brick — plaster both sides directly on brick surface | 31.6 | 48.8 |
| Wood and Sheet Rock — studding filled with porous gypsum | 12.3 | 44.8 |
| Wood and Gypsolite — studding filled with porous gypsum | 11.8 | 42.4 |
| Wood and asbestos — wood studs, ½″ asbestos on one side, joints filled | not given | 27.4 |
| Wood and Celotox — wood studs, ½″ Celotex on one side, joints filled | not given | 26.7 |

## UNIVERSITY OF WASHINGTON

**Tested at only three frequencies (288, 576 and 1152)**

| Material in Test Panel | Relative Amount of Sound Transmitted, | Relative Audibility of Sound Through Panel |
|---|---|---|
| | *Col.* 1 | *Col.* 2 |
| 3″ Cinder Block — ½″ plaster, both sides | 917 | 1.00 |
| 3″ Plaster Block — ½″ plaster, both sides | 870 | 0.992 |
| 2″ Solid Plaster — on channels and metal lath | 741 | 0.978 |
| 2¼″ Solid H. W. Plaster — ¾″ channels and metal lath | 632 | 0.960 |
| 3″ Clay Tile — ½″ plaster, both sides | 588 | 0.953 |

NOTE.— The University of Washington report does not list reduction factors, or give rating in decibels. Column 1 lists the relative amount of sound transmitted through a panel. The values are to an arbitrary base; panel with a higher coefficient transmits more sound than one with a lower coefficient.

Column 2 lists the relative audibility of sound transmitted through a panel. Since this is on the basis of sensations produced on the ear, the coefficient values are rated according to the logarithms of the intensities of sounds transmitted through a panel. The higher the coefficient, the more sound that is heard through a panel. In other words, of the five panels tested, clay tile is best and cinder block poorest for sound insulation.

In considering the reduction factor required, two things should be known: (1) the intensity of the sound which it is desired to reduce, and (2) the intensity of sound present in the room. Very slight noises in a room usually mask any transmitted sound having a resultant intensity of 20 or less sensation units. To reduce a noise having an original intensity of 70 sensation units (loud speech or noisy stenographic room) to inaudibility in a fairly quiet room, would require a partition wall having a reduction factor of 50. If any other noises are present in the room, their intensity may be such as to mask other transmitted sounds having a resultant intensity greater than 20 sensation units. For instance, normal street noises present in the average business office will completely mask other sounds having intensities of as much as 30 to 40 sensation units. It is readily understood, therefore, that the intensity of noises present in a room is an important factor in determining the requirements for sound insulation in the partition walls.

The following table lists the average intensities of noise levels present in typical buildings:

| Type of Building | Average Intensity of Noise Present (decibels) |
|---|---|
| Radio Studios | 20 |
| Churches, Libraries | 30 |
| Residences, Hospitals | 30 |
| Theatres, Auditoriums | 35 |
| School Rooms | 40 |
| Business Offices | 45 |
| Hotel Lobbies | 50 |
| Department Stores | 60 |

In order that a transmitted sound be inaudible, a partition wall should have a reduction factor which will reduce the sound to an intensity of 10 to 15 decibels less than the values listed above.

Experiments on some walls composed of two wythes with a 2-inch air space, show an increase in the reduction factors ranging from 30 per cent to more than 100 per cent over that recorded for one wythe of the same material. We have no record of similar experiments with brick masonry, but it is reasonable to suppose that two wythes of brick on edge (2¼-inch) or laid flat (3¾-inch) with a 2-inch air space between would have reduction factors at least 30 per cent greater than that given for a single wall. In other words, a double wall of brick on edge or flat, with a 2-inch air space, might easily be expected to have reduction factors ranging from 65 to 75 decibels.

The reduction factor of most partitions may be increased by floating the partition; i.e., placing padding material at the top and bottom of the partition. Such construction eliminates practically all vibration in the partition that might otherwise be communicated directly to it by the floor or ceiling. It might be added, however, that a floating floor and suspended ceiling aid materially in deadening or reducing sounds.

## 312. THERMAL EXPANSION

The coefficient of thermal expansion of brick masonry varies, depending upon the coefficients of both the brick and the mortar. An average value may be taken as 0.000004 in. per in. per degree F.

Theoretically the expansion of a solid brick wall 100 ft. long due to a temperature change of 70 to 100° F. might be expected to cause serious cracking if expansion joints were not provided to take up the movement of the wall. However, this theory has not been borne out by actual experience. Many such walls which have been exposed to severe temperature changes over a period of many years, show no evidence of cracks which might be attributed to expansion. The reason for this apparent contradiction of the theory has not been determined; it may be due to compensating movements of the masonry units and mortar, to the compressibility of the mortar, or to a combination of several factors.

Field observations of parapet walls have disclosed cracks which in some instances might be attributed to expansion either of the parapet wall or the roof slab.

It is recommended, therefore, that the corners of all parapet walls be reinforced with a minimum of one ¼″ round rod in every third joint continuous around the corner and extending into the masonry at least 3 feet and that parapet walls exceeding 50 feet in length should be reinforced for their entire length.

As indicated above, expansion joints are not required in solid brick walls; however, when such walls are supported upon a foundation containing expansion joints it is recommended that the brick masonry be adequately reinforced over the expansion joint to prevent cracks in the masonry due to lateral movement of the foundation.

## 313. WETTING BRICK

Emphasis has been placed in other sections of this book on the importance of a low rate of absorption or penetrability of brick when laid. Fig. 6, Chapter 2, shows the relation of bond strength to penetrability between brick and identical mortars having three different flows.

Based upon these data and the results of numerous other investigations, it is recommended that brick having natural rates of absorption (penetrabilities) of 20 grams per min. or less when placed flat side down in ⅛″ of water be laid dry and that brick having rates of absorption in excess of 20 grams per min. be wet sufficiently so that their rate of absorptions when laid does not exceed 20 grams per min.

## 314. CONSTRUCTION DURING FREEZING WEATHER

Brick masonry can be built during temperatures below freezing (32° F.); however, laying brick, particularly those requiring wetting, in freezing weather is not recommended.

If brick are laid in temperatures below freezing, all materials, brick and mortar materials, should be heated to a temperature such that they will remain above 32° F. until after they have been placed and suitable protection has been provided. Brick having rates of absorption above 20 grams per min. should be wet and after the brick are laid the finished masonry should be protected against freezing for a period of at least 48 hours.

## 315. WORKMANSHIP

To secure the desired performance of brick masonry, good workmanship is essential and should not be sacrificed for speed of production. A skilled artisan will, however, produce good workmanship at relatively high speed, for his knowledge of the essentials automatically guides his technique.

In the Bureau of Standards investigation of wall strength (BSRP 108), two classes of workmanship were employed to determine their effect upon wall strength, and similar procedure was followed in other tests for moisture penetration, etc. These two classes were called "Ordinary" and "Inspected" workmanship, which terms will be used in other chapters as reference to the following descriptions.

"Ordinary" workmanship may be considered as that very closely approximating the quality of work generally obtained in commercial construction where close supervision is not obtained and the first consideration is economy. This class of workmanship is characterized by the furrowing or grooving of mortar beds, very little mortar in the end joints, and almost no mortar in the collar (vertical longitudinal) joints. This type of workmanship showed lower strengths and less satisfactory results in practically all investigations than better workmanship.

"Inspected" workmanship is that which may be obtained under a definite specification, followed by careful supervision. In this class of workmanship the mortar beds are spread smooth or only slightly furrowed, the ends of the brick are buttered with sufficient mortar to completely fill the end joint when the brick is in place, and the vertical longitudinal joints are completely filled by slushing. The same results may be obtained by pouring the interior vertical joints full of grout, by the "pick and dip" method or by "shoring", the object being to obtain a solid masonry mass entirely free from voids or open joints.

The above classifications pertain to the assembling of materials into a structural mass of brick masonry. Workmanship also includes other functions of great importance in the stability and appearance of brick structures. Brick should be laid on level beds and to plumb vertical lines. Wythes should be well bonded by headers, or as otherwise required. Architectural patterns and surface bonds should be carefully laid out in advance and carefully executed in the work, and horizontal and vertical joint thicknesses should be determined to work to openings and corners with the minimum of variation.

Bricklayers should, at all times, keep in mind the requirements of the other trades in order to build in or make proper provision for the installation of the other branches of the work. All such built-in work should be solidly bedded in mortar or grout.

The quality of the materials used has its effect upon workmanship. A highly plastic mortar works easily and therefore tends to improve the class of workmanship and at the same time increase the rate of bricklaying.

## 316. BONDS, STRUCTURAL AND PATTERN

The word "bond" in the structural sense means "to bind" and, therefore, refers to the method of arranging the brick units so that, by their overlapping, the entire mass of masonry is thoroughly tied together into a unit. This "structural bond" naturally results in the formation of a pattern on the exposed face of the masonry, which is also called "bond". The term bond may also refer to the adhesion of mortar to brick and to steel reinforcement.

Structural bond is obtained in ordinary masonry by means of headers every fifth, sixth or seventh course. In particular bond patterns, the equivalent number of full headers should be used and any additional headers required by the pattern may be half brick. If the exposed face is to show stretchers only, structural bond may be obtained by means of blind headers, which require a great deal of clipping. Structural header bonding may be omitted but the work must be well done and all joints completely filled with mortar or preferably grout to obtain the best adhesive bond. Metal ties are used frequently to bond masonry wythes together.

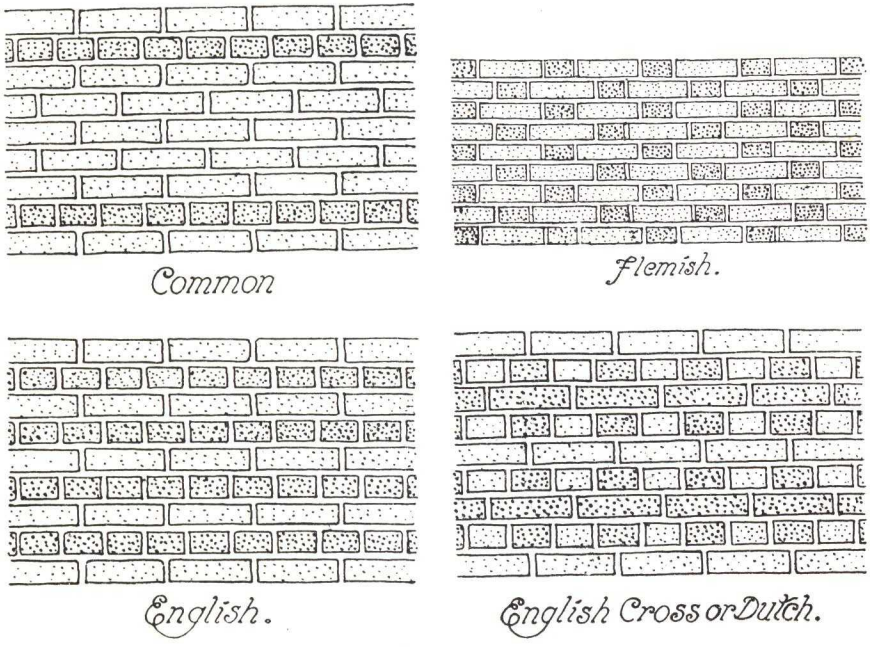
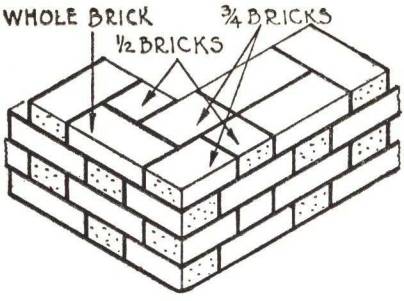



*Figure 11*
*Typical bond patterns.*

Bond patterns most commonly used are illustrated in Figure 11. These patterns should be carefully laid out and constructed. Methods of building corners are shown in Figure 12 and it is important that if "closures" of clipped brick are used, they should never be placed directly at a corner, but should have at least a full header starting the course. These bonds may be varied or emphasized by the use of different colored brick or by changing the type of exposed joints. Geometrical patterns may be formed of brick, but they should be carefully designed to be appropriate and in proper scale with the elevation.



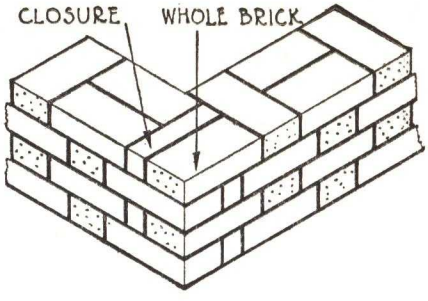




*Figure 12*

*Dutch corner bond.* *English corner bond.*

### 317. EXPOSED JOINTS

Joint thickness affects to some extent the strength of the wall. Although no definite relationship has been proven, walls with thin joints tend to have a somewhat higher strength. For standard brick, a ½″ joint is most useful in forming patterns and bonds, since two headers plus the joint exactly equal the length of the stretcher.

The color, section and texture of joints will affect to a marked degree the interest and quality of the finished wall. Color of the joints should be kept uniform despite gradations in the brick shading. Dark colored mortar tends to subdue shadows and deepens the tone of the wall. Light or natural colored mortar gives a play of brilliant shadows.

Texture of the joint may resemble that of the brick or contrast with it, and is controlled by the use of a steel or wood surfacing tool, and the use of coarse sand or fine gravel in the mortar mix. Four types of exposed joints are illustrated in Figure 13. Tooled joints (types 3 and 4) which compress and spread the mortar after it has set slightly, produce the best weathering properties.

(1) **Weathered Joint.** This is formed as a plain cut joint, finished with the trowel after the mortar has slightly stiffened. Each course of brick will throw a horizontal line of shadow along the wall. It is a water-shedding, low-cost joint, much to be preferred over the struck joint (not illustrated).

(2) **Flush or Plain Cut Joint.** This is formed by cutting surplus mortar from the face of the wall. If a rough texture is desired, the joint must not be manipulated with the trowel. For an extremely rough joint, the surface may be tapped with the end of a rough cut piece of wood after the mortar has slightly stiffened.

(3) **"V" Joint.** This is similar in method of forming and performance to the Concave Joint. It should be formed with a special tool, but may roughly be made with a square-edged board, rubbed at an angle along the joint.

(4) **Concave Joint.** This is perhaps the best joint and is formed with a special tool or a bent iron rod. It is weather resistive and inexpensive.

(5) **Struck Joint.** This is the most simply formed of all joints and is widely used for interior walls. Its use for exterior walls is not recommended, however, because its weather resistive qualities are distinctly inferior to the other joints illustrated.

### 318. ARCHES

The building of arches requires careful workmanship. The skewbacks should be cut properly and solidly built first. Every brick used in the arch should be fully bedded in

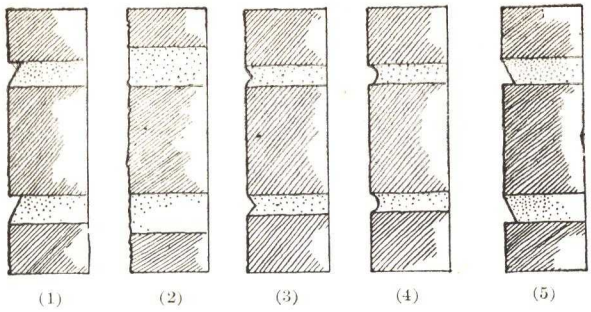



*Figure 13*
*Types of exposed joints.*

mortar on all sides by shoving all brick into place and not by feathering the ends or sides and attempting to fill joints by slushing. The key (of brick or other material) should be small enough to allow a full mortar bed all around. In cutting the brickwork over an arch (to fit the extrados), have all pieces so cut as to permit of a solid mortar joint of uniform thickness with the arch bricks.

In short, the arch should have solid supports and solid masonry (with no voids) in the arch itself and the brickwork immediately above.

### 319. TOOTHING

Toothing should not be permitted if its use can be avoided, but it may be used in joining new work to old. When used, see that all joints in the toothing are filled, not just feathered or pointed on the outside. Careless work at such points has sometimes resulted in wall leaks.

### 320. ANCHORS

Wall anchors and other ties are set by the brickmason and should be solidly bedded in mortar. It may be well to check also their number and location to insure compliance with the plans and specifications, or the provisions of the local building code, referring to their uses.

### 321. CHIMNEYS AND FIREPLACES

In the construction of these parts, careful workmanship is most essential. All joints must be full, in order that the chimneys be air tight. The space between flue linings and brickwork must be completely filled with mortar as the work progresses; flue linings must precede brickwork as the chimney is built up and not pushed down on the inside of brickwork already built. Careless work around a fireplace may cause faulty operation of a well-designed assembly.

More specific design data and construction details are given in Chapter 6.

### 322. WALL OPENINGS

It is the duty of the brickmason to see that all brickwork supports over wall openings are properly placed and on a proper bearing. Lintels, beams and similar parts should have end bearings laid in a full bed of mortar and on solid brickwork (not hollow). If flashings or other waterproofing are called for, the brickmason should see that they are installed as specified.

## 323. CARE DURING CONSTRUCTION

Materials should be so stored as to avoid the absorption of excessive moisture. Unfinished walls should be covered at night with canvas or tar paper. Reinforced brickwork or concrete floors, when built simultaneously with walls should be so constructed that the wash from their surfaces does not come in contact with the wall faces.

## 324. MAINTENANCE

The proper maintenance of brick masonry is important in the prevention of efflorescence. Downspouts and gutters should be kept in repair. Cracks, due to settlement or other causes should be promptly pointed up. The appearance of efflorescence at any point may be an indication of faulty workmanship, which should be promptly corrected.

## 325. CLEANING CLAY PRODUCTS MASONRY

Processes used in cleaning depend primarily upon the materials to be cleaned and the nature and chemical composition of the spots or stains to be removed. Cleaning clay products is no exception to this general rule. A method which can be used successfully with glazed ware for instance, may prove entirely unsatisfactory if applied to a rough textured brick wall. The problem is further complicated by the fact that masonry structures consist of two materials, the clay units and the mortar, each of which has different characteristics — many cleaning compounds to which clay products are entirely impervious have a damaging effect upon mortar.

The texture and absorption* of the clay unit and the nature of the material or stain to be removed also determine to a very great extent the results which may be obtained with various cleaning methods. Proper cleaning methods will restore glazed ware to its original appearance, the same is true of smooth textured brick or tile of relative low absorption, the lower the absorption the better the results which may be expected.

High absorption or rough textured brick are more difficult to clean. If the staining material has penetrated into the pores of the clay unit, it frequently cannot be removed without removing a part of the unit itself, thus destroying the original texture and resulting in an appearance different from the original. The texture of "sand struck" or sand finished brick, is produced by a thin layer of sand which adheres to the clay. This sand frequently burns to a different color than the body of the brick and if it is removed by cleaning, both the texture and color of the unit may be altered.

The principal methods of cleaning masonry structures are: sand blasting, steam or steam and water jet and the application of various cleaning compounds. The first two of these methods are used principally on large buildings as considerable equipment is necessary for either method. Cleaning compounds may be applied to either large or small structures.

### (a) SAND BLASTING

Sand blasting consists in blowing hard sand through a nozzle by compressed air against the surface to be cleaned. The sand removes a thin layer from the wall surface, the thickness of the layer depending upon the depth to which dirt or stain may have penetrated the wall. This method is an effective cleaning process, however, it destroys the original texture of the unit and leaves the wall with a coarse texture which is particularly susceptible to the accumulation of soot and dirt. Due to the difference in hardness between clay units and the mortar joints, sand blasting may do serious damage to the joints.

* Porosity, absorption, capillarity and rate of absorption are all factors which affect the cleaning of clay products. In this discussion the term absorption should be understood to include all of the above.

If sand blasting is used it will frequently be necessary to repoint the mortar joints after the surface has been cleaned; and the application of a colorless waterproofing compound to the roughened surface will tend to make the wall self cleansing, and will prevent the rapid soiling of the surface from smoke and dust particles in the air. A waterproofing compound containing paraffin should be used for the purpose.

D. W. Kessler of the Bureau of Standards states as conclusion No. 4 in Technologic Paper No. 248: "The treatments giving the highest waterproofing values and appearing to be most durable are those which use paraffin as the waterproofing element, either alone or in conjunction with other materials. The deterioration or loss of waterproofing value on materials of this type is not appreciable within a period of two years."

Sand blasting should never be used on glazed ware or other units having special surfaces or textures.

### (b) STEAM OR STEAM AND WATER JETS

This method of cleaning consists of washing the wall with a steam or steam and water jet under pressure. It is effective in removing soot and dirt which accumulates on a wall over a period of time. Best results are obtained when it is used on glazed ware or low absorption units, however, it has also been used with fair success on high absorption or rough textured brick. It is not effective in removing stains which have penetrated into the pores of the brick nor in removing such substances as mortar or paint which may adhere to the brick.

Frequently an alkaline such as sodium carbonate, sodium bicarbonate or trisodium phosphate is added to the cleaning water when the cleaning is done by the jet method. Chemically these compounds are known as salts and while they aid materially in the cleaning action some of the salt will be retained in the clay unit (the amount depending largely upon the absorption of the unit) and will appear later on the face of the wall in the form of efflorescence. The amount of the salt retained by the clay units can be materially reduced by thoroughly wetting the surface with clear water before the cleaning solution is applied. The wall should also be washed with an abundance of clear water after cleaning to remove the salt from the surface.

### (c) APPLICATION OF CLEANING COMPOUND

This method is applicable to structures of all sizes and is probably used to a greater extent than either of the other methods. On large projects the cleaning contractor develops a cleaning compound best adapted to the particular job. This is done through an examination and analysis of the material and stains to be removed and the strength and chemical composition of the solution is usually adjusted by trial.

### (d) TREATMENT OF SURFACE PRIOR TO CLEANING

Most cleaning solutions contain compounds which if absorbed by the clay units will subsequently appear on the face of the wall in the form of efflorescence. For this reason the surface to be cleaned should be thoroughly wetted with clear water before the cleaning solution is applied. A booklet —"Maintenance of Interior Marble" by D. W. Kessler — which is based upon the results of our 10,000 laboratory tests and experiments, contains the following statement: "The importance of thorough rinsing is so evident and has been so frequently stressed that it hardly seems worth while to emphasize the point here. However, the research upon which many of the recommendations herein are based has indicated that the final rinsing is no more important than the preliminary dampening of the surface with clear water. Where this practice is followed the final rinsing can be more easily and satisfactorily done."

## (e) TREATMENTS FOR REMOVAL OF STAINS

### (1) Mortar Stains:

Remove chunks of mortar from wall with putty knife or cold chisel. Soak wall with clear water. Wash with dilute solution of hydrochloric or phosphoric acid. If hydrochloric (muriatic) acid is used, do not use a stronger solution than one part acid to ten parts of water; with the phosphoric acid use one part acid to twenty parts water. The acid solution will attack the mortar joint hence it is important that the finished work be thoroughly washed with clear water immediately after cleaning.

### (2) Paint Stains:

For fresh paint apply a commercial paint remover, or a solution of tri-sodium phosphate in water — two pounds of tri-sodium phosphate to one gallon of water. Allow to stand and remove paint with scraper and wire brush. Wash with clear water.

For very old dried paint, organic solvents similar to the above may not be effective, in which case the paint must be ground off by sand blasting or scrubbing with steel wool.

The following recommendations are taken from "Maintenance of Interior Marble" by D. W. Kessler, or "Removing Stains from Cast Stone and Concrete — Concrete Building and Concrete Products Vol. 12, No. 3, March 1937" by R. E. Baumgarten. It is believed that they may also prove effective with clay masonry.

**Iron Stains.** — Mix seven parts lime-free glycerine with a solution of one part sodium citrate in six parts lukewarm water, and make with it a thick paste of whiting or kieselguhr. Apply paste to stain with trowel, and scrape off when dried out. Repeat until stain has disappeared and wash thoroughly with clean water. Other suggestions are given for particularly bad cases. — R. E. Baumgarten.

**Tobacco Stains.** — Dissolve two pounds tri-sodium phosphate in five quarts of water. In separate vessel make a smooth stiff paste of twelve ounces of chloride of lime in water. Pour the former onto the paste and stir thoroughly. When the lime has settled, draw off the clear liquid and dilute with equal parts of water. Make a stiff paste of this with powdered talc and apply in the same way as described above for iron stains. —R. E. Baumgarten.

**Smoke Stains.** — Make smooth stiff paste of tri-chlorethylene and powdered talc and apply as described above. Cover with glass or pan to prevent rapid evaporation. If slight stain is left after several applications, wash thoroughly and then use the method described under tobacco stains. Precaution should be taken to ventilate a closed space in which tri-chlorethylene is used, as the fumes are harmful.—R. E. Baumgarten.

**Copper or Bronze Stains.** — Mix together in the dry form one part of ammonium chloride (sal ammoniac) and four parts of powdered talc. Add ammonia water and stir until a thick paste is obtained. Place this over the stain and leave until dry. When working on polished marble use a wooden paddle to scrape off the paste. An old stain of this kind may require several applications. Sometimes aluminum chloride is used in the above procedure instead of the sal ammoniac.—D. W. Kessler.

**Oil Stains.** — Make a paste of a solution of one pound of tri-sodium phosphate to one gallon of water and whiting which may be obtained at any paint store. Spread this paste in a layer about ½ inch thick over the surface to be cleaned and leave until it dries (approximately 24 hours). Remove the paste and wash surface with clear water.—D. W. Kessler.

## CHAPTER 4

# FOOTINGS, FOUNDATIONS, PIERS AND COLUMNS

### 401. FOOTINGS

Brick masonry has a long and satisfactory history in its use as footings for all sizes and types of structure. Brick is not affected by alkalis or acids and may be used in any type of soil.

Among other advantages, the use of brick masonry for footings and foundations permits more uniform and continuous progress on the job. Brick footings may be expeditiously started as soon as a portion of the bearing surface has been prepared, eliminating the necessity for protecting the trenches, building and removing forms and additional cleaning.

Brick footings are constructed of either plain masonry or reinforced brick masonry as required by the structure and the foundation bed. Only the plain brick masonry footings will be discussed in this chapter.

### 402. BEARING VALUES OF SOILS

The purpose of footings is to distribute the entire load of the structure over soil areas, sufficiently large and of the proper proportions to produce equal soil pressure throughout. The pressure must not be in excess of the bearing value of the foundation bed and unit pressure of each footing in a structure should have the same relation to the bearing capacity of the respective supporting sub-soils. In other words, when a structure is supported by footings which rest upon material of the same bearing capacity, the unit load per sq. ft. should be the same on each footing. If the nature of the bearing capacity of the sub-soil varies under a particular structure, the footing should be designed accordingly. This is necessary to prevent unequal settlement.

The bearing capacity of soils depends upon their composition, the degree to which they are confined and the amount of moisture which they contain. In the absence of test data on the bearing capacities of different foundation beds, the values given in the following table may be used as a guide:

TABLE NO. 20

BEARING CAPACITIES OF VARIOUS SOILS

| Foundation-bed | Tons per sq. ft. | Pounds per sq. ft. |
|---|---|---|
| Soft clay | 1 | 2000 |
| Wet sand | 2 | 4000 |
| Firm clay | 2 | 4000 |
| Sand and clay, mixed or in layer | 2 | 4000 |
| Fine and dry sand | 3 | 6000 |
| Hard, dry sand | 4 | 8000 |
| Coarse sand | 4 | 8000 |
| Gravel | 6 | 12000 |
| Soft rock | 8 | 16000 |
| Hard-pan | 10 | 20000 |
| Medium rock | 15 | 30000 |
| Hard rock | 25 | 50000 |

The bearing values of various soils have been given to indicate their relative merits in a traditional sense. These values have been used for years in localities where experience has verified their accuracy within reasonable ranges of safe loading. There are many kinds and mixtures of soils, and it is not always possible to judge the safe bearing capacity of any given soil by mere reference to a table such as that given above. When there is any doubt, more accurate information should be obtained.

The most common method of checking the bearing capacity of a sub-soil is by means of borings and load tests. Pre-determined areas are loaded and the settlements observed. It is a well-known fact, however, that a small area will bear a larger load per sq. foot for a short time than a larger area will perpetually. It is important, therefore, that the tested area be as large as practicable and that the test continue for several weeks.

The modern science of soil mechanics has come to be recognized as the proper approach to foundation problems. In this, the type of soil is considered by the study of such factors as the size and arrangement of the grain particles. The lessons of experience, however, are not tossed aside, and consideration is also given to the depth of the foundation material and nature and use of the structure, the allowable settlement and effect of time. It is important, therefore, that foundation structures of considerable magnitude and importance should have the bearing value of the supporting soils investigated, in order to obtain the more accurate data.

For the average size building, the designer may obtain additional information regarding the bearing capacity of soils in any particular locality by conferring with the local building commissioner. It is also probable that the local building authorities have established values for the bearing capacity of soils in that vicinity.

## 403. ECCENTRIC LOADING

An eccentric load is one whose line of action does not coincide with the gravity axis of the supporting member. An eccentric load on a footing increases the unit compressive stress on the side nearer the eccentricity and decreases the unit compressive stress at the opposite side of the footing. The standard formula for obtaining the unit stresses due to an eccentric load upon a rectangular footing or piers of homogeneous material is as follows:

$$F_b = \frac{P}{A}\left(1 \pm \frac{6e}{d}\right)$$

$F_b$ is the total unit pressure at outer edge of footing.
P is the total load applied.
A is the cross-sectional area of footing.
e is the eccentricity of the load, measured from center of footing or pier.
d is the total lateral dimension of the footing in the direction of the eccentricity.

The equation with the plus sign gives the maximum unit stress at the edge near the eccentric loading, while the equation with the minus sign is used to obtain the unit stress at the far edge. When the result is positive, the stress is compression; when negative, the stress is tension.

A study of the above formula will indicate that an eccentricity (e) less than $\frac{d}{6}$ will result in compression over the whole bearing area of the footing; an eccentricity equal to $\frac{d}{6}$ will result in zero pressure at the far edge increasing to double the average unit pressure at the edge near the eccentric loading; when the eccentricity is greater than $\frac{d}{6}$, the resultant unit stress at the far edge is negative and is, therefore, in tension.

In order to avoid a negative pressure at the far edge of the footing, it is important that the footing be designed and placed with reference to the applied load, so that the resultant of the loads will pass through the middle third of the lower surface of the footing. This is an essential requirement unless special provisions are made for the effects of the eccentric loading on both the member supported and the foundation-bed. Unless the foundation-bed is of rigid material, excessive bending stresses may be produced in the members supported and it may be necessary to provide reinforcement to resist these stresses. Where eccentric loading cannot be avoided, the wall or column and the connection with the footing should be made both strong enough and stiff enough to resist the tendency of the footing to tip without excessive deformations. Since the movements usually are resisted by horizontal forces acting on the wall or column, provision should be made to transmit these forces (usually through the first floor construction to cross walls) to parts of the structure able to resist them.

### 404. MINIMUM DEPTHS

In locations subject to freezing temperatures, footings should be carried below the frost line in order to avoid disturbance from the freezing and thawing of the foundation material.

Alternate wetting and drying disturb most soils. It is essential, therefore, that the footings be placed at a reasonable depth whether or not there is frost action. Most local building codes stipulate a minimum depth for footings.

### 405. ANCHORAGE

Wherever there is a possibility of load applied to a wall or column, which may cause it to be lifted from the footing, such members should be anchored to the footing. Anchorage usually will be found necessary in buildings of light occupancy subjected to winds of hurricane intensity. An investigation of the anchorage requirements should be made when designing buildings of less than four stories if the height is greater than twice the least width, and for higher buildings when the height is greater than three times the least width.

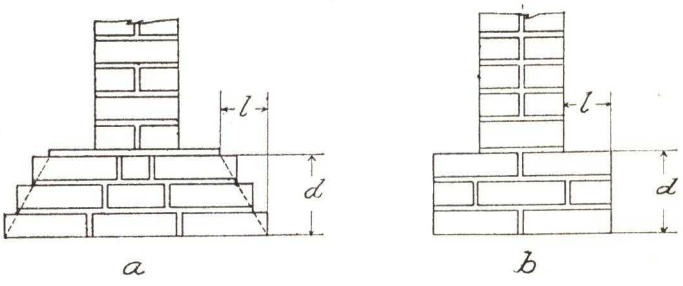
### 406. DESIGN OF FOOTINGS

Footings should be designed to carry the dead and live loads without exceeding the bearing capacity of the supporting soil as described in Section 402. In buildings subject to unequal live loads, the footings should be proportioned accordingly. In most buildings, however, the live load is assumed as of uniform magnitude over the whole area of the building. The pressure on the foundation of a tall building should be considerably less per unit area than that which may be allowed for a lower structure. A slight inequality of bearing and consequent unequal settling of a tall building might imperil the safety of the entire structure, while in the case of a lower building, the result is not so serious.

The areas of the lower surfaces of footings are determined by the requirements relating to pressures on foundation beds and the depth (thickness) and width at other elevations should be proportioned to avoid bending and shearing stresses exceeding the allowable working stresses. It may be considered that this requirement has been met if the corbelling is made in accordance with the recommendations contained in this section.

Brick masonry footings are increased to the necessary size by offsets (corbelling) from the pier, column or foundation wall. The projection beyond that portion of the masonry above acts as a cantilever beam loaded by the reaction of the soil or footing below. The proper offset for each course depends upon the vertical load and the transverse strength of the masonry course. Where transverse strength is an important factor, it is recommended that the footing be designed in reinforced brick masonry.

Brick footings should always start with a double course at the base and then for ordinary loads, offsets may be made in each course as laid. For heavy loads the corbelling or offset is made in every second course with the lower being a stretcher course and the upper a header course. A corbelled course of header brick should project not more than 2″, and if the outer brick in a footing course are laid as stretchers, it is recommended that the corbelling be not more than 1″.




The permissible working stress on the top of a pier or column footing is obtained by the following formula:

$$P_b = 0.25\, f'_b \left(\frac{A}{A'}\right)^{1/3}$$

in which $P_b$ equals permissible working stress over the loaded area, lbs. per sq. in.

A equals total area of top of footing, sq. in.

A′ equals loaded area at top of footing, sq. in.

$f'_b$ equals ultimate compressive strength of brickwork, lbs. per sq. in.

This limits the area of the upper bearing surface, and the requirements relating to pressures on foundation beds fix the lower area.

## 407. FOUNDATION WALLS

Foundation walls usually serve as both retaining walls and bearing walls. As ordinarily built, they would often be unstable except for the superimposed loads and the supports received from floors and cross walls.

The stability of foundation walls depends upon their resistance to vertical loads and to lateral earth pressures as well as upon their durability. Estimates indicate that the maximum compressive stresses in foundation walls of dwellings, produced by vertical loads, do not exceed 60 lbs. per sq. in. of gross area for walls 8 in. or more in thickness. Inasmuch as most codes require the use of combinations of masonry units and mortars, which with good workmanship are likely to result in walls having strengths at least five times that value, it is apparent that the stresses produced by the vertical loads alone need not be considered.

Most masonry structures are designed on the assumption that the masonry is incapable of resisting tensile stresses. This is a fallacy because an examination of the stresses which may be produced by lateral earth pressure indicates that tensile stresses may occur in foundation walls. It appears that the requirements for masonry foundation walls of dwellings should be based largely upon the factors which affect the stability of the wall when subjected to lateral pressures.

The fact that the countless number of these foundation walls remain stable and have not ruptured is an indication that the brick masonry has sufficient tensile strength to resist such tensile stresses as may ordinarily be produced by lateral earth pressures. It has also been shown by test data in Chapter 3 that brick masonry walls do have some

tensile strength and are, therefore, capable of resisting to that extent any tensile stresses that may develop.

Occasionally building codes have recognized this fact and have made some allowance for tensile strength in brick masonry. It has not been customary, however, to design plain masonry foundation walls on the basis of a theoretical stress analysis. Most building regulations fix the thickness of such walls in accordance with the experience and common practice which has been found safe for that particular locality.

There is one feature which many building codes have apparently overlooked, and that is the fact that the larger the vertical load on a foundation wall, the greater the stability of that wall. For instance, a foundation wall supporting a masonry superstructure is more stable than the same thickness of foundation wall supporting a frame or other type of lighter superstructure.

Most designers will readily recognize this fact, but it is also apparent from a study of the charts in Section 409. Nevertheless, some building regulations require a greater thickness for a foundation wall supporting a masonry dwelling than for a foundation wall supporting a similar frame structure.

There is no justification for this requirement. It is sometimes claimed that the additional weight of the masonry superstructure creates sufficient additional compressive stresses in the foundation wall to warrant the additional thickness. This is also a fallacy, because, as stated earlier in this section, the compressive stresses produced by vertical loads on foundation walls of dwellings are seldom more than one-fifth the allowable compressive strength for brick masonry which is permitted in most building codes.

## 408. DESIGN AND ANALYSIS OF FOUNDATION WALLS

Most foundation walls of residential, store and small apartment buildings, whose thicknesses are not arbitrarily fixed by local building regulations, are designed by a sort of rule of thumb. The thicknesses are usually determined by convention based upon the results of experience. A recommended method or rule is that a plain brick masonry foundation wall will be sufficiently stable to resist vertical loads and lateral pressures and maintain its position on the footing, if the foundation wall:

(1) is at least as thick as the wall supported,
(2) is not greater in height between footing and first floor than 12 times the thickness,
(3) supports a dead load equivalent to the weight of a solid wall of the same thickness and at least 12 feet in height.

In addition to the above conditions, the designer is reminded that lateral support in the horizontal direction obtained by cross walls, piers or buttresses will increase the stability of the foundation wall. If the height approaches the maximum of 12 times the thickness, it is advisable to provide such lateral support at intervals not exceeding 18 times the wall thickness, or preferably to reinforce the wall.

There are occasions, however, when it is desirable to make a design of a foundation wall based on a theoretical analysis.

In the design analysis of a foundation wall, all the vertical loads are determined as well as their point of application on that wall. In the absence of other data, brick masonry may be assumed as weighing 125 lbs. per cu. ft. The earth pressure is usually assumed as equivalent to that produced by a liquid weighing between 15 and 45 lbs. per cu. ft. The lower values are used when the soil is firm and cohesive, while the higher values are assumed in wet or saturated soils. The values of 20 to 30 lbs. per cu. ft. are more often used. If "w" represents the weight per cu. ft. of the equivalent fluid and "h" the height of the foundation in feet, the lateral pressure on the wall varies from zero at the surface to "wh" at the base and the total resultant earth pressure, $wh^2/2$, is applied normal to the wall at one-third the height from the bottom.

In the stress analysis of a foundation wall the following six arbitrary assumptions as to manner of support, constitute a range within which the actual construction will probably fall:

(1) The wall is considered as a slab simply supported at the footing and first floor; neglecting the lateral supporting effect of end walls, piers, cross walls, etc.

(2) The same as (1), except that the wall is assumed as fixed at the footing;

(3) The same as (2), except that the wall is also assumed as continuous in a vertical direction at the first floor line;

(4) The wall is considered as a vertical slab panel, simply supported at the four edges; at the footing and first floor and by end walls, piers or cross walls;

(5) The same as (4), except that the wall panel is assumed as fixed at the footing;

(6) The same as (5), except that the wall panel is also assumed as continuous in a vertical direction at the first floor line.

There are other possible combinations of assumptions regarding the degree of fixity at the footing and continuity with adjoining masonry at other supports; but the above six conditions will afford a fairly complete study.

Of course, design analyses are made for only as many of the six conditions as may apply to a particular foundation wall. The first (1) is probably the most severe and computations based on those assumptions will usually indicate greater tensile stresses than those resulting from the other five assumed conditions.

The maximum computed stress will occur at the point of maximum moment. This critical point may be found by plotting the moment curve for each of the six conditions. However, it may not be necessary to plot the moment curves, except in the analysis of an unusual foundation design. The location of the critical point or points is influenced by the amount and location of both vertical and lateral loads. In most foundation designs, it will be found that the maximum moment will occur in the inside face at approximately mid-height of the backfill. In studies 2, 3, 5, and 6, an investigation should also be made of the outside face at the elevation of the footing; while studies 3 and 6 may require an additional check at the outside face opposite the first floor.

Inasmuch as the assumed conditions 4, 5, and 6 consider the wall as a rectangular slab supported at the four edges, the critical points are analyzed for moment in both the vertical and horizontal direction. All of the vertical loads may be taken into consideration in the determination of moment for the vertical element. The lateral earth pressure, however, may be considered as distributed in both vertical and horizontal directions. The proportion of the load or pressure carried by the vertical element is $\frac{V^4}{H^4+V^4}$, where V is the vertical dimension (height) of the wall panel and H is the horizontal dimension. In a like manner, the proportion of the load carried in the horizontal direction is $\frac{H^4}{H^4+V^4}$ The sum of the two should equal the total load or pressure. For convenience, there is a chart in the appendix which shows the proportionate distribution of loads for rectangular slabs supported at four edges.

The resultant stress at any point is not the sum of the two stresses due to vertical and horizontal moments. In fact, according to the theory of Poisson's ratio for lateral strains, axial stresses of the same type tend to decrease the resultant stress at a point. However, to be on the safe side, if the computed stresses in both horizontal and vertical

directions are of the same kind, it is recommended that the greater of the two be considered as the resultant maximum stress.

Moments are taken about any point in question and the corresponding stress at that point is then determined by means of the equation:

$$S = \frac{6M}{bd^2}$$

Where, S is the stress in lbs. per sq. inch at the extreme fiber.

M is the computed moment in inch lb., d is depth of slab (thickness of wall) in inches and b is width of unit section in inches.

It is seldom that a compressive stress will be a matter of concern, but cognizance should be given to any indication of tensile stresses. If the latter are found to be excessive, the wall should be re-designed to increase its stability by additional thickness or less distance between supports. It is frequently more economical to design a reinforced brick masonry wall.

The maximum calculated tensile stress will depend on the assumption as to the manner of support of the wall and also on the magnitude of the assumed earth pressure. If an earth pressure equivalent to a fluid pressure of 20 or 30 lbs./sq. ft. is assumed, an analysis of many foundation walls which have given satisfactory service for years will indicate that relatively high tensile stresses are developed in these walls. The ultimate tensile strength of brick masonry depends upon the strength of the mortar and to a very great degree upon the workmanship or manner of laying the brick. For high tensile strength it is very important that the rate of absorption of the brick when laid be as low as possible without the brick floating on the mortar bed. Low rate of absorption may be obtained with highly absorbent brick by thoroughly wetting immediately before laying. With strong mortar, brick having low rates of absorption and first-class workmanship (all mortar joints completely filled) tensile strengths of brick masonry in excess of 100 lbs./sq. in. may be obtained. However, in determining a safe permissible tensile stress for any locality, it is suggested that the calculated stresses in existing constructions be determined as a check on the assumed earth pressure, and that these stresses in conjunction with the expected ultimate tensile strength of the masonry be considered in fixing an allowable tensile stress. In many localities it will be found that assuming an earth pressure equivalent to a fluid pressure of 30 lbs./sq. ft., relatively high tensile stresses up to 80 lbs./sq. in. may be justified for the most unfavorable assumptions as to the manner of support.

## 409. RESULTS OF ANALYSES OF SMALL RESIDENTIAL FOUNDATION WALLS

In order to have valid comparisons of the relative stability of typical small residential foundation walls, a stress analysis has been made of the foundation conditions normally found. The structure was assumed as a one story, four-room house. The floor, partitions and roof were of light wood frame construction. The stresses were calculated for both 8 and 12 inch foundation walls supporting either brick or frame exterior walls. The following table indicates the eight different conditions assumed for analysis:

Both the footing and the floor have been assumed to provide rigid supports against horizontal movements of the foundation wall at those levels. The lateral supporting effect of the end walls on the maximum stresses was neglected. A pier or cross wall placed approximately midway between the end walls would have reduced the computed tensile stresses.

The greatest tensile stress in a masonry foundation wall with a given horizontal thrust is obtained with the least vertical load. In this instance, the joists extend in the north and south direction and only the girder exerts its smaller reaction on the east and

west walls. Therefore, the east wall was selected for analysis as one bearing the lesser weight. A summary of assumed dead loads is shown on Figure 14. The dead loads for Cases V and VI are not shown, but were the same as Cases I and II, respectively.

**CONDITIONS ASSUMED FOR STRESS ANALYSIS OF FOUNDATION WALLS**

| Case | Thickness of Wall | Type of Exterior Wall Supported | Lateral Support of Basement Foundation Walls |
|---|---|---|---|
| I | 12 in. | 8 in. brick | Simply supported at footing and 1st floor. |
| II | 8 in. | 8 in. brick | Same as Case I |
| III | 12 in. | 8 in. brick | Simply supported at footing and continuous with superstructure. |
| IV | 8 in. | 8 in. brick | Same as Case III |
| V | 12 in. | 8 in. brick | Fixed at footing and simply supported at 1st floor. |
| VI | 8 in. | 8 in. brick | Same as Case V |
| VII | 12 in. | 6 in. frame | Same as Case I |
| VIII | 8 in. | 6 in. frame | Same as Case I |

The intensity of the lateral pressure of the earth against the foundation wall was assumed to be equivalent to that produced by a liquid weighing 20 lbs. per cu. ft. This was selected for illustrative purposes only. It is less than values commonly used in the design of foundation walls when no information is available as to the nature of the soil. It may be greater than the pressure which would be exerted by dry cohesive soils but is much less than many of the values obtained experimentally in investigations of the lateral pressures of earth fills.

Relations between the maximum tensile stresses in the basement wall selected for analysis and the height of backfill are shown in Figures 15, 16 and 17. Figure 15 shows these relations for an 8-in. basement wall which supports brick masonry walls above grade, each of the four curves indicating values for a different degree of restraint at the footing on first floor or location of maximum tensile stress. Similar data for basement walls 12 in. in thickness are shown in Figure 16. Although calculations of the same kinds were made for basement walls supporting a wood frame construction, only the data for Cases VII and VIII are shown in Figure 17. No useful information beyond that shown in Figures 14 and 17 were obtained by the other calculations.

The graphs show the stresses at the sections of greatest bending moment. For walls simply supported at top and bottom, the maximum tensile (or minimum compressive) stress was at the inside face near mid-height of the backfill. With the base fixed, it was at the outside face at the footing level. For walls considered to be continuous with the superstructure, this maximum stress was at the inside face near mid-height of the backfill, although there was also a relatively large stress in the outside face at the first floor level for both the 12 and 8 in. foundation walls.

The maximum tensile stresses in the foundation walls with brick masonry superstructure are less than in the foundation walls with frame superstructure, as can be seen by comparing Figures 15 and 16 with Figure 17. For the 8 in. foundation walls simply supported at the footing and first floor levels, the maximum tensile stress is 6 lb./sq. in. less with the heavier superstructure for the maximum height of backfill. The addition of another story on the dwelling with exterior walls of brick would decrease the maximum tensile stress in an 8 in. foundation wall at mid-height on the inside face by approximately 9 lb./sq. in. and about 6 lb./sq. in. in the outside face at the footing level.

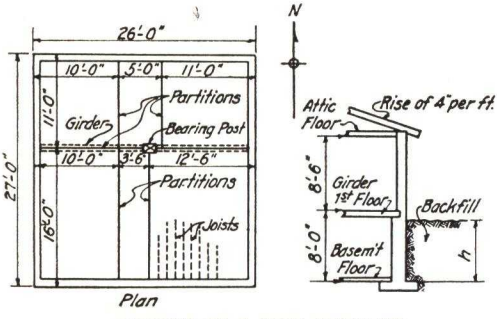
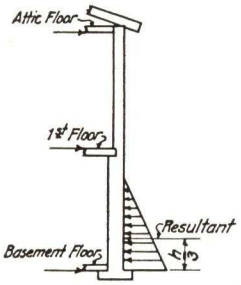
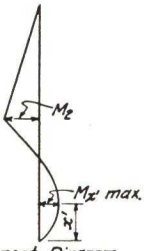



DIAGRAM OF 4-ROOM DWELLING

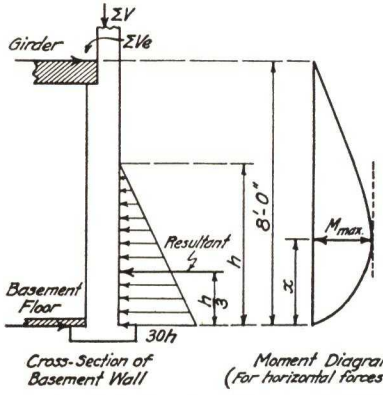



Cross-section




Moment Diagram (For horizontal forces only)

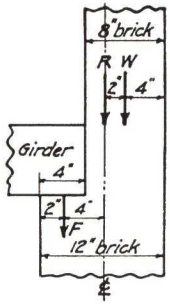
BASEMENT WALL CONTINUOUS WITH SUPERSTRUCTURE




Cross-Section of Basement Wall

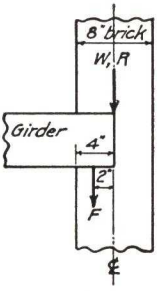
Moment Diagram (For horizontal forces only)

BASEMENT WALL SIMPLY SUPPORTED



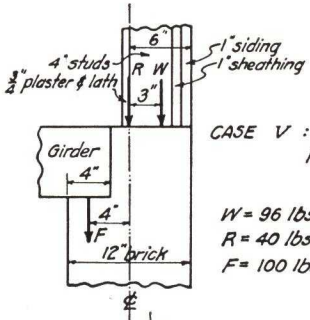

CASES I and III : 8" brick wall on 12" brick basement.

W = load due to weight of wall = 590 lbs. per linear foot.
R = load due to weight of roof = 40 lbs. per linear foot.
F = load due to weight of floors and partitions = 100 lbs. per linear foot.



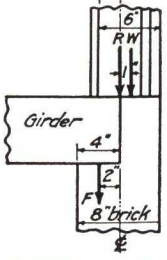

CASES II and IV : 8" brick wall on 8" brick basement.

W = 590 lbs. per linear foot.
R = 40 lbs. per linear foot.
F = 100 lbs. per linear foot.




CASE V : 6" frame wall on 12" brick basement.

W = 96 lbs. per linear foot.
R = 40 lbs. per linear foot.
F = 100 lbs. per linear foot.




CASE VI : 6" frame wall on 8" brick basement.

W = 96 lbs. per linear foot.
R = 40 lbs. per linear foot.
F = 100 lbs. per linear foot.

VERTICAL LOADINGS ON BASEMENT WALL

*Figure 14*

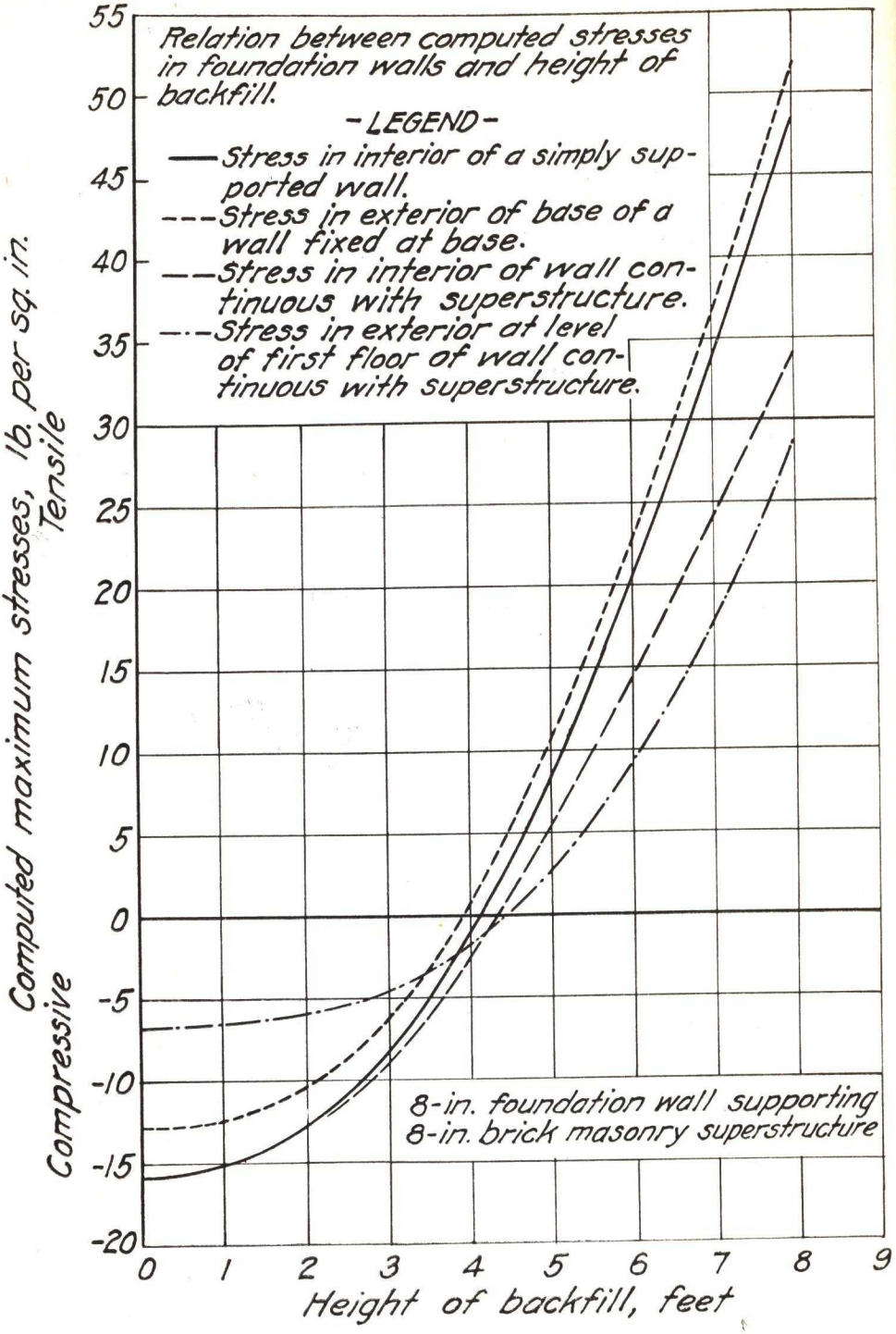

Relation between computed stresses in foundation walls and height of backfill.
- LEGEND -
—— Stress in interior of a simply supported wall.
--- Stress in exterior of base of a wall fixed at base.
—— Stress in interior of wall continuous with superstructure.
—·— Stress in exterior at level of first floor of wall continuous with superstructure.
8-in. foundation wall supporting 8-in. brick masonry superstructure
Computed maximum stresses, lb. per sq. in.
Tensile
Compressive
55
50
45
40
35
30
25
20
15
10
5
0
-5
-10
-15
-20
0
1
2
3
4
5
6
7
8
9
Height of backfill, feet

*Figure 15*

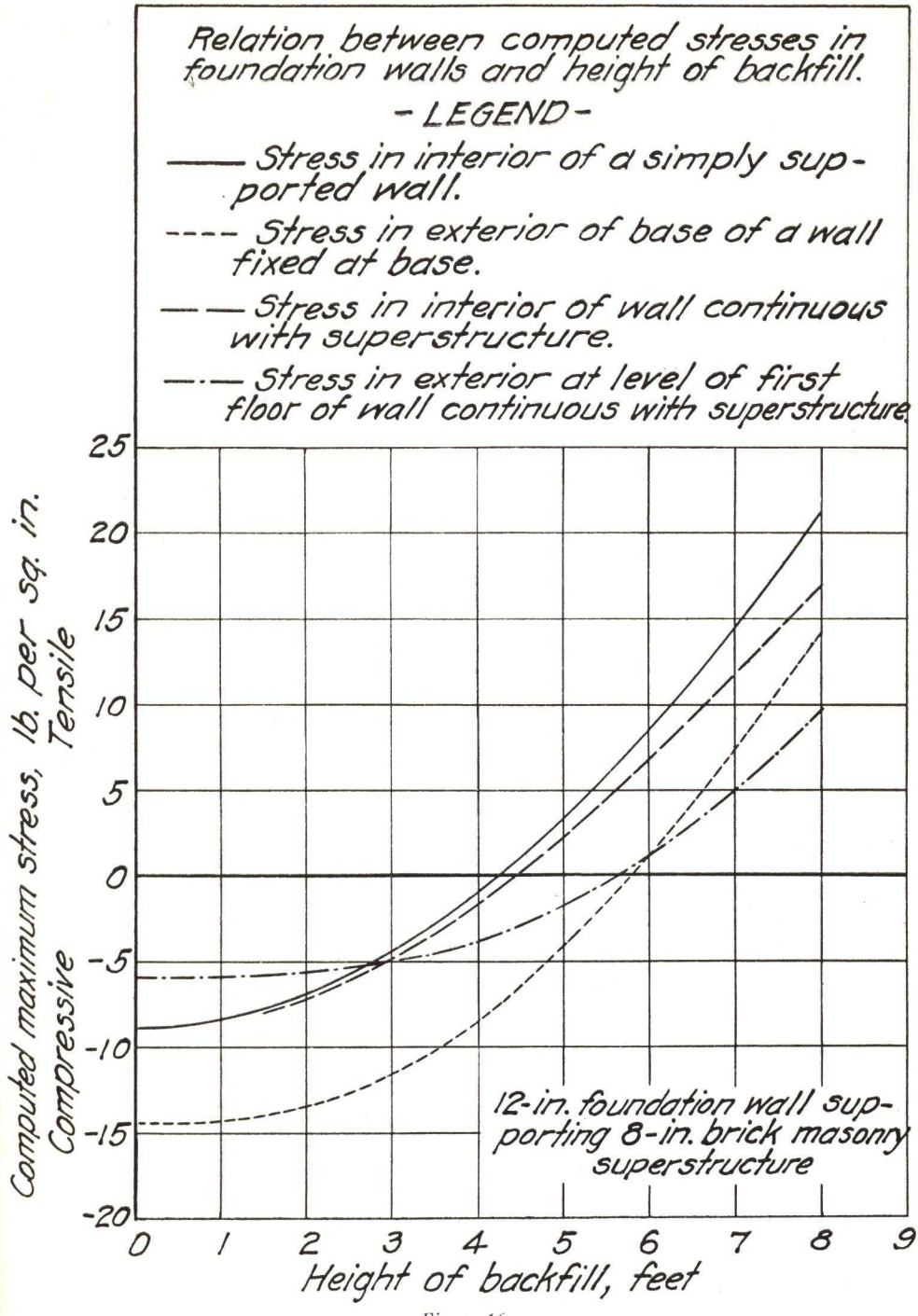

Relation between computed stresses in foundation walls and height of backfill.
- LEGEND -
—— Stress in interior of a simply supported wall.
---- Stress in exterior of base of a wall fixed at base.
— — Stress in interior of wall continuous with superstructure.
—·— Stress in exterior at level of first floor of wall continuous with superstructure
Computed maximum stress, lb. per sq. in.
Tensile
Compressive
25
20
15
10
5
0
-5
-10
-15
-20
0
1
2
3
4
5
6
7
8
9
Height of backfill, feet
12-in. foundation wall supporting 8-in. brick masonry superstructure

*Figure 16*

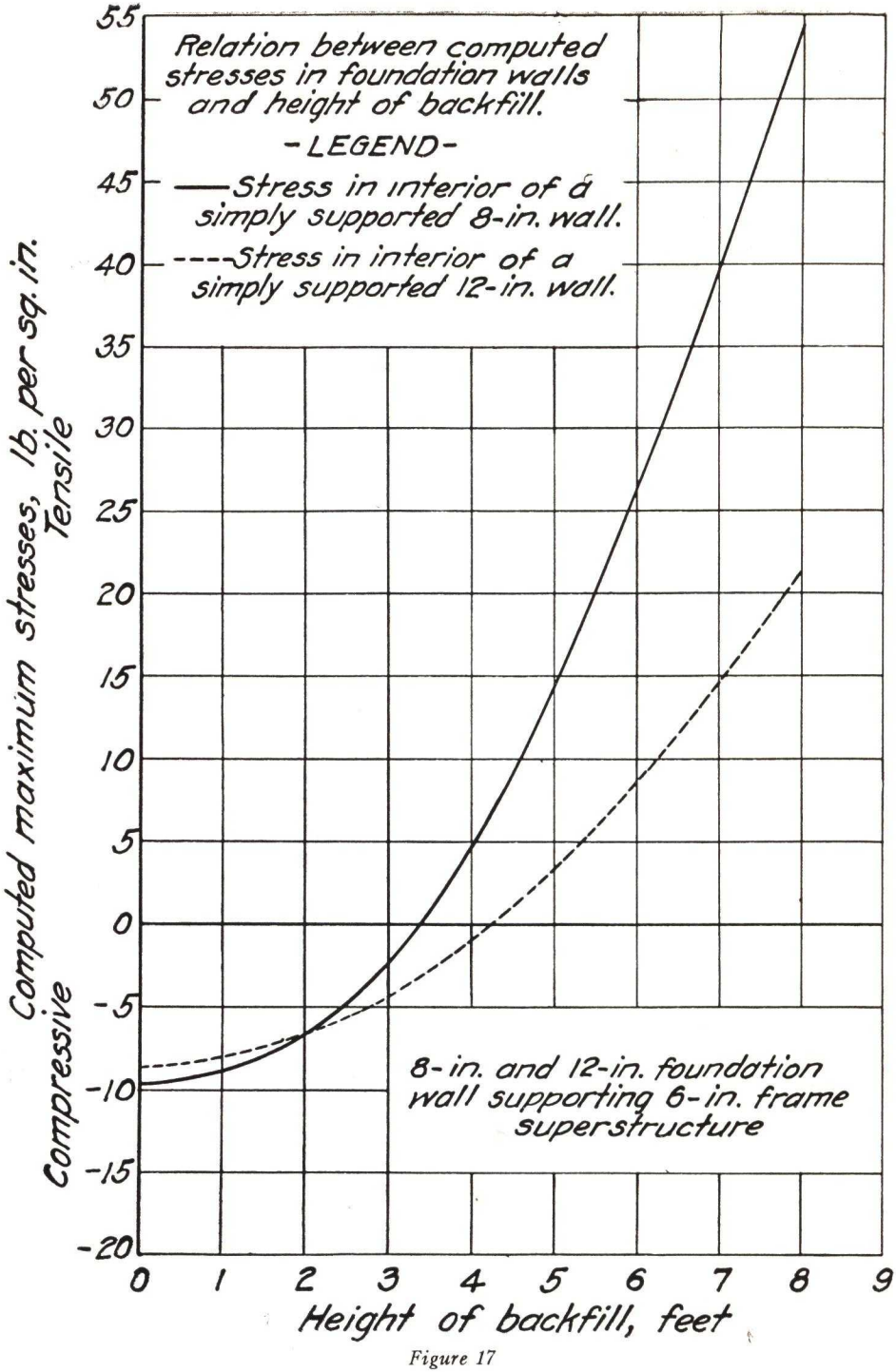

Relation between computed stresses in foundation walls and height of backfill.
-LEGEND-
——Stress in interior of a simply supported 8-in. wall.
----Stress in interior of a simply supported 12-in. wall.
8-in. and 12-in. foundation wall supporting 6-in. frame superstructure
Computed maximum stresses, lb. per sq. in.
Tensile
Compressive
55
50
45
40
35
30
25
20
15
10
5
0
-5
-10
-15
-20
0
1
2
3
4
5
6
7
8
9
Height of backfill, feet

*Figure 17*

## 410. DRAINAGE

It is advisable in almost all cases to place a porous tile drain at the bottom of all exterior basement walls or footings to carry off any water which may accumulate. This drain should be 4″ to 6″ in diameter and laid with a slope in the direction of flow of not less than ⅛″ per foot, the high point of which is not above the floor level and the low point not below the bottom of the wall or footing. The tile should be carefully laid on a solid bearing with no dips or traps and with the ends butted together and connected to the drainage outlet with vitrified tile in sealed joints from the building to the connection. The butted joints in the drain tile around the foundation should be not more than ¼″ apart and wrapped with a strip of copper mesh or other rust-proof gauze, approximately 4″ wide, to allow water to seep into pipe at joints without carrying sediment.

The fill over the tile should be of dry material such as coarse gravel, crushed stone or broken brick placed carefully — not dumped — to avoid breaking the tile. The fill over the tile should be graded from coarse at the bottom to the fineness of sand at the top of the fill.

## 411. HOLLOW FOUNDATION WALLS OF BRICK

Hollow walls of brick may be confidently used for basement foundation walls when working stresses are not exceeded and conditions of stability are satisfied. There are several types of hollow brick masonry walls with masonry bonding units which have been used successfully for a number of years. In some of these walls, the hollow space is utilized for conduit space and also as the cold-air return in a hot air heating system. The same precautions regarding waterproofing of solid walls apply to hollow walls of brick.

## 412. PIERS AND COLUMNS

Individual piers, as differentiated from those serving as pilasters, etc., in masonry walls, are defined as compression members having a ratio of unsupported length to least width of less than six. Piers may be designed on the basis of working stresses for plain brick masonry.

According to the test data and conclusions of Prof. W. J. Krefeld, Columbia University, reported in A.S.T.M. Proceedings, Vol. 38, Part I, pp. 363-369, there is a tendency for the strength of brick masonry to increase as the slenderness ratio decreases below six and to remain practically constant from six to twelve. These tests indicated that the allowable working stresses for brick masonry should be multiplied by the correction factors corresponding to the slenderness ratios listed below.

### TABLE NO. 21

### CORRECTION FACTORS FOR VARIOUS SLENDERNESS RATIOS

**Multiply the normal allowable compressive strength of brick masonry by the factor corresponding to the slenderness ratio**

| Slenderness Ratio (height/thickness) | 1.5 | 2.0 | 3.0 | 4.0 | 5.0 | 6.0 | 8.0 | 10.0 | 12 |
|---|---|---|---|---|---|---|---|---|---|
| Strength Correction Factor | 1.7 | 1.5 | 1.25 | 1.12 | 1.04 | 1.00 | 0.97 | 0.94 | 0.92 |

A column may be defined as a compression member whose unsupported length is more than six times its least width. Basement columns of plain brick masonry should not have a slenderness ratio greater than ten. If in such columns there is a possibility of bending due to eccentric loads, or in the event that the slenderness ratio is greater than ten, the columns should be designed as reinforced brick masonry.

## 413. WORKMANSHIP

**(a) No part of the building is more important than the footing.** If there is the least doubt regarding the firmness of the soil, the excavation should be carried down to firmer ground, and the footing made wider by stepping off with more courses. Never place a wall on filled ground or spongy, springy soil. The lowest course in a footing should rest on a full bed of mortar spread over the subsurface to a depth of not less than ½″, and preferably 1″. Every other course from the bottom upward and the outer brick in each course should be headers. All joints should be completely filled with mortar or grout. If the supporting material happens to be rock, all loose and decayed portions should be cut away and the bed dressed to a plane surface as nearly perpendicular to the direction of the pressure as is practicable. A sloping rock surface should be stepped to prevent or avoid the tendency of the footing to slide.

**(b) Foundations.** — Foundation walls should be properly bonded and kept plumb with all courses horizontal. All joints should be completely filled and tooled in accordance with all requirements for good masonry. Maximum strength is thus developed, as well as water tightness. The outside of foundation walls below grade should be parged, as the work proceeds, with a coat of cement mortar (1:2 mix), well troweled on and not less than ½-inch thick, to an even smooth surface. In damp or wet soils, it is advisable to add a coating of bituminous waterproofing.

**(c) Piers and Columns.** — The construction of piers and columns is governed by the same precautions as foundations, except that damp-proofing is not necessary. All work should be plumb, well-bonded and joints horizontal. Joints should be completely filled with mortar or grout. After the mortar has time to set partially, the mortar joints should be tooled with a round jointer to produce a concave joint.

# CHAPTER 5

# WALLS AND PARTITIONS

## 501. LOADS

The permissible loads on bearing walls may be determined in the same manner as for piers. The thickness of walls, however, is usually arbitrarily established by building codes and the actual load per sq. inch is seldom more than one-tenth of the allowable compressive strength of the brick masonry. The strength requirements are well insured by "inspected" workmanship and proper bonding with headers or ties every sixth course or by the use of grouted masonry without through headers.

## 502. THICKNESS OF BEARING WALLS

The information now available on the effects of the qualities of the component materials on the properties of brick work makes it possible to design walls to meet any of the usual service requirements. By combining this information with the principles of engineering design, walls may be constructed with assurance that they will fulfill their intended functions.

The thickness of walls may be influenced by several factors but usually will be controlled by the following:

(a) **Limitations on compressive stresses.** If the effects of eccentric loading and lateral forces are taken into account in computing these stresses, the working stresses may be taken as given in Chapter 9.

(b) **Limitations on thickness required** to resist lateral forces (Discussed in Sections 505 and 506).

(c) **Limitations required for fire resistance** (Discussed in Chapter 3).

(d) **Considerations of thermal conductivity** (Data on conductivity are also given in Chapter 3).

## 503. ECCENTRIC LOADS

The effect of eccentricity of loading on the compressive strength of brick work is approximately the same as would be estimated from calculations based on the assumption that sections plane before the loading would remain plane after the loading. Since the agreement between theory and test results has been so well established, it is evident that the effects of eccentricity of loading may be estimated closely by calculations which apply strictly only to members composed of a homogeneous, isotropic and elastic material. This provides the only general method for estimating the maximum stresses in load bearing masonry walls.

The chief drawback to the accurate calculation of stresses in eccentrically loaded walls, several stories in height, lies in the labor involved in the calculation of moments. The problem must be solved by one of the methods applicable to stress analysis of statically indeterminate structures (such as the theorem of three moments, slope deflection method, Costigliano's method, etc.). Because of the gain in accuracy, greater assurance of safety and better economy in the use of materials, the expenditure of the labor involved may be justified whenever a single calculation may serve for the design of an appreciable portion of the wall volume in the structure.

A far more common practice is to make certain assumptions regarding the nature of the structure in order to reduce the problem to one in which the moments may be determined from statical considerations alone. The usual assumptions are as follows:

1. The wall behaves as if it were not continuous but pin connected at each floor level, being restrained at the piers against horizontal translation only.

2. The maximum moment within any story height is equal to the vertical load of the floor next above, times the eccentricity of its line of action.

Such analyses as have been made indicate that the errors introduced by these simplifying assumptions usually are small and are on the side of safety.

In certain cases, it is common to make allowances for eccentricity of loading, empirically, by lowering the working stresses. Generally, this is a vicious practice because, if the allowance is a safe one for some cases, it must necessarily be grossly uneconomical for others. Experienced designers may follow this practice with satisfactory results provided they have made sufficient calculations to have learned what reductions are suitable for different types of structures and loadings.

An arbitrary reduction in working stresses to cover all types of structures and loadings cannot be justified; though such a requirement or a similar one specifying minimum wall thickness may be entirely satisfactory if applied to but one well-defined type of structures (such as houses).

## 504. LATERAL STRENGTH

Obviously a plain masonry wall alone, of the dimensions ordinarily employed in buildings, would not be stable if subjected to lateral forces from either wind storms or earthquakes of only moderate intensities. Their well known resistance to such forces is due to the closely spaced supports from the floors and intersecting walls. The thickness of walls required will, therefore, be a function of (1) the magnitude of the lateral forces, (2) the spacing and arrangement of the lateral supports and (3) the inherent strength of the masonry.

The assumed wind pressures or horizontal accelerations due to earthquakes used in design should properly be a matter for local decision. In rare cases, an assumed maximum wind pressure of 15 pounds per square foot of vertical projection probably is adequate, but in others, assumed pressures as high as 30 pounds per square foot would not be excessive. Similarly, the magnitude of lateral forces from earthquakes assumed in design should depend upon the prevalency of earthquake damage in the locality. Where long records are not available, it is suggested that an assumed wind pressure of 15 lbs. per sq. ft. for those portions of the building less than 60 ft. above ground and not less than 20 lbs. per sq. ft. for those portions more than 60 ft. above ground, and an assumed horizontal earthquake acceleration of one-tenth that of gravity be used in design, i.e., about 3 ft. per sec. per sec.

## 505. WIND RESISTANCE

In specifying minimum wall thickness for lateral stability against wind pressures, it is convenient to distinguish between walls with openings and those without, and to classify them further in accordance with their ratios of length to width. For walls without openings, the design for resistance to lateral forces may follow the methods used for rectangular slabs supported along either two or four edges. In Table 22 are given the maximum permissible ratios of unsupported length to thickness for load-bearing or curtain walls, in which:

$l$ = length of shorter span between lateral supports, in feet,
L = length of longer span between lateral supports, in feet,
t = thickness of wall in feet.

TABLE NO. 22

MAXIMUM VALUES FOR $l/t$

**(Based on assumed maximum wind velocity of 60 miles per hour. For greater wind velocities multiply values in table by $\frac{60}{V}$, where V = assumed wind velocity in miles per hour)**

| Values of $l/L$ | Maximum Values of $l/t$ | | | |
|---|---|---|---|---|
| | For Mortars Equal in Strength to Cement or Cement - Lime Mortars | | For Lime Mortar | |
| | Top Story | Lower Stories | Top Story | Lower Stories |
| Less than 0.5 | 18 | 20 | 13 | 14 |
| From 0.5 to 0.8 | 20 | 22 | 15 | 16 |
| From 0.8 to 1.0 | 24 | 26 | 17 | 19 |

For walls with large openings, usually it is an economical practice to design the masonry portions extending between floors as vertical beams. This is particularly true in load-bearing walls because the vertical loadings add to the stability by tending to prevent the formation of tensile stresses in the vertical direction. The moments are calculated by assuming the wind pressures to be uniformly distributed over the entire panel area and the panel to be supported at the floor levels. On account of the uncertainty of the full continuity of the walls, the safest practice is to ignore the effect of continuity in estimating moments. By adding algebraically the stresses due to the vertical dead loads and the maximum tensile stresses due to moments, the resulting maximum stress is found. The maximum compressive stress is the sum of that due to the vertical dead and live load and the maximum stress due to moment. The design is safe if: (1) there are no tensile stresses and (2) if the maximum compressive stress is not greater than that allowed.

## 506. EARTHQUAKE RESISTANCE

In designing structures to resist earth tremors, it is necessary to determine the forces involved separately from those due to wind pressures. The horizontal forces due to earthquakes are calculated in the same manner (except in direction) as are dead loads due to gravity. The force acting on any mass is assumed to be a fractional part (usually 1/10) of the weight. Vertical accelerations are neglected because of their lower intensity and because structures are better able to withstand them.

The design of brick masonry walls to resist earthquake stress must necessarily be based on some assumptions. There is quite general agreement at the present time with Japanese doctrine that it is uneconomical to design structures for resistance against maximum possible forces (which may not occur even during the life of the structure) but against forces of average known intensity. And these have been placed at about those due to an acceleration of 1/10 gravity, or about 3 feet per second per second.

According to the well known formula of mechanics, the maximum force is the product of mass times this acceleration, the mass in most cases being the total weight of the structure at or above the point of assumed application of the force divided by the gravitational constant.

A complete discussion of this involved subject would require too much space, but certain considerations, as they may apply to brick masonry construction, are stated herewith.

Failure of brick masonry from earthquake force may be caused by a force normal to the wall face or parallel thereto. It has been determined by calculation and by field observation that the outer walls of the more common types of masonry buildings, which are braced and supported laterally by cross walls and floors, have sufficient strength in bending and shear to resist forces due to the acceleration named. In brick panel or filler walls, if presumed to lend stiffness and rigidity to the structural framing, the required strength is that developed in the plane of the wall, usually assumed to be the plane of the neutral axis or the center of gravity.

Careful analyses indicate that resistance to forces in this plane requires compressive strength and shearing strength in the masonry panel. This latter is probably a function of the shearing strength of the mortar joints. Even though it may do so, a brick panel wall should not be called upon to completely resist distortion of the horizontal and vertical framing members, for diagonal bracing should be provided in these latter members, in the design.

No extensive laboratory tests have been made to determine the resistance of brick panel walls to distortion in the plane of the wall, but from data on racking tests (National Bureau of Standards BMS5, Nov. 1938) as well as the compressive strength of masonry and shearing strengths of mortar joints, calculations of considerable accuracy indicate that brick panel walls, properly built into the framing, add a high degree of rigidity to the structure and are quite unlikely to be ruptured under normal conditions.

The work of the Japanese investigators and engineers as well as those who have studied this problem on our own Pacific Coast, may be consulted for detailed analyses and treatment of this involved problem.

In addition to the design of walls for strength, consideration should be given to the possibility of resonance between the vibrations of earthquakes and the natural periods of the structure and its component members. The latter phase of the problem is frequently overlooked in earthquake design. Authorities agree that as long as the natural period of a structural member is less than 1.0 second there will not be set up serious vibrations.

For story heights of from 9 to 11 feet and brick walls with cement or cement-lime mortar, no investigation is needed; but with lime mortar, the periods may easily approach the critical value of one second. Rectangular walls may be considered safe in this respect, if they do not support appreciable weights between the floor levels and if

$$\frac{0.3L^2}{t\sqrt{E}} \text{ is greater than } 1.0$$

where, L equals spans between supports (floors), in inches,
t equals thickness, in inches, and
E equals modulus of elasticity of wall.

## 507. LATERAL SUPPORT

In the paragraphs immediately preceding, frequent mention was made of the lateral support afforded to walls by floors and intersecting walls. The discussion was based upon the assumption that no floor or cross wall would be considered as a support unless:

1. There was positive anchorage between the wall and the supporting structure.
2. The floor system had sufficient stiffness as well as strength to transmit the forces without serious distortion to cross walls or other members which would resist them, and
3. The intersecting walls would transmit the forces directly to the foundation without undue deformation.

The need for positive anchorage of walls to floors is readily apparent when it is considered that horizontal forces from wind or earthquakes may act both inward and outward on the walls. The friction between, say wooden floor joists and supporting walls, is not sufficient anchorage even for very moderate wind pressures.

Common types of wall anchors, for tying floor joists and roof plates to walls are described in Section 528.

Provision should also be made for horizontal thrusts in a direction at right angles to that of joists. Adjacent joists, parallel to walls, should be suitably tied thereto at intervals not exceeding 8 feet. Such ties should be continued through intermediate joists unless bridging is employed throughout the floor system.

Floors do not always constitute adequate lateral supports for walls unless they are stiff enough to transmit the forces from the walls to rigid cross walls, continuous to the foundation, and without causing excessive deformation in the walls supported. Floors of reinforced concrete, with or without hollow tile fillers, or of reinforced brickwork undoubtedly have the required stiffness.

The distance which floors may safely be depended upon to transmit lateral forces is influenced by the width of the floor and by the material of which it is built. Safe practice indicates the following ratios to be safe for ordinary buildings, but in very narrow buildings the ratios should be reduced and for very wide buildings they may be increased: for wood joist construction, 60 times the minimum wall thickness; for steel frame, fire-resistive construction, or mill or slow-burning construction, 90 times the minimum wall thickness.

Intersecting walls of solid masonry without openings are ideal for transmitting horizontal forces from outside walls to the foundations. When, however, the walls consist largely of windows or other openings between columns or piers, they are of little use in that connection. Special provisions must be made in such cases. Diagonal bracing is most effective and may be readily provided in brickwork unless it interferes with the openings.

## 508. CONCENTRATED LOADS

When concentrated loads from beams, girders or similar members are carried by brick masonry, provision for spreading the load must be made if unit working stresses are exceeded. But, since such concentrated stress may safely be assumed to spread through a supporting pyramid of masonry whose sides slope not less than 30° to the direction of load application, allowable working stresses in the masonry directly under a concentrated load may safely be twice that permitted in the same masonry as a whole. For example, if the 6-inch flange of a beam had a bearing of 8 inches on a brick wall, the 48-square inch area would support a reaction of 48 times double the allowable working stress for the particular type of brick masonry.

When the load reaction exceeds this amount, steel plates or similar members should be inserted to spread the load over sufficient bearing area.

In hollow walls of brick, or combination walls of brick and hollow units, two or more courses of solid brick should be built in to provide a bearing for concentrated loads. In most constructions, these solid courses are in the form of belt courses at this elevation.

## 509. SUPPORT OVER OPENINGS

The dead weight of a brick wall which must be supported over openings can safely be assumed as the weight of a triangle whose height is one-half the actual span of the

opening. The corbelling section of the brickwork on both sides, above the top of the opening, distributes the superimposed load over the opening to the masonry at the sides.

The more common method of supporting brickwork over openings is by steel angles or other structural shapes. However, reinforcing rods, laid in the mortar beds of the brickwork, will take the tensile stress and form a reinforced brick masonry lintel or beam. For complete information and data see Chapter 10 on R-B-M beams and lintels.

**Brick Arches.** — Various forms of segmental and flat arches are also used to carry brickwork over openings. The stresses developed in various types of arches, and the reactions at the haunches, are determined by the more common methods described in standard works on structural mechanics.

A common rule for segmental arches of brick is, to build them with a rise of one inch for each foot of span. The thrust at the springing line for uniform loading may be determined with sufficient accuracy by use of the following formula:

$$\text{Horizontal thrust} = \frac{\text{load on arch (pounds) x span (ft.)}}{8 \text{ x rise of arch (ft.)}}$$

For a concentrated load at the center, the thrust will be twice as great.

Brick arches are sometimes built of radial bricks, moulded or ground to proper dimensions; but more often standard brick are used, the wedge-shaped mortar joints compensating for the curvature.

Wooden centering for support during construction is the more common form.

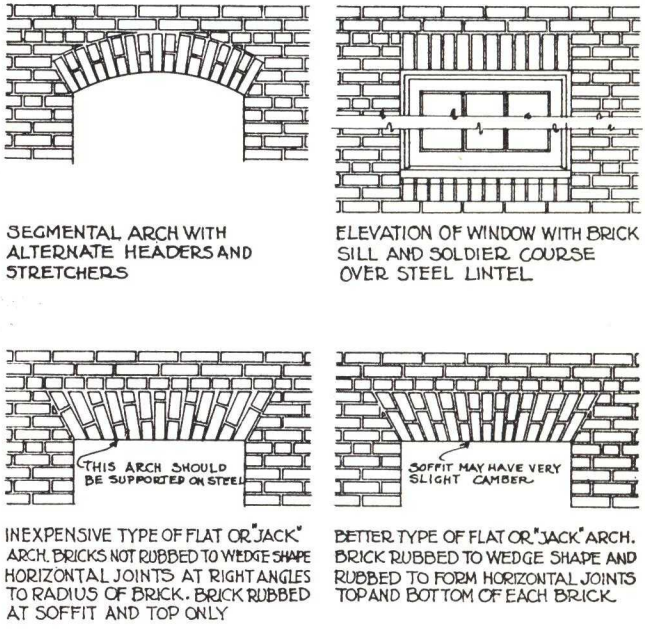



*Figure 18*

*Typical arches and lintels.*

## 510. CURTAIN AND PANEL WALLS

Although panel walls lack the stabilizing effect of vertical loads acting on load-bearing walls, this advantage is more than offset when the panel walls are built between and in contact with girders or columns. The arching effect of the end restraint tends to greatly increase their resistance to lateral forces, the arch action being particularly effective for walls having a high modulus of elasticity.

The larger panels, whose area is determined by the spacing of the framing members, should be able to resist the maximum lateral and compressive stress likely to occur, even though built of only 8-inch thickness of solid brickwork. Hollow or combination panel walls may need greater thickness.

The general provisions applying to outer walls, apply equally well to panel and curtain walls. In addition, see that courses are properly laid out and the work is so executed that the periphery, or boundary, of all such walls is built solidly against the supporting framework. When the facing course projects beyond the framing proper, angle or other supports should be used, and the facing, in all cases, properly bonded to the backup.

It is usually necessary, and most always desirable to build flashing or other water-diverting members into spandrel walls. The mason should see that this is not neglected.

## 511. PARAPET WALLS

The stability of parapet walls is an important consideration because of the danger to life in case of damage during wind storms or earthquakes. Some codes require that the height of parapet walls of brickwork be not greater than four times the thickness. Apparently this requirement has been sufficiently rigid in localities not subject to high winds or to frequent earth tremors. Experience in California and Florida indicates strongly that it is not sufficiently rigid for those locations, and damage by tornadoes in Omaha, St. Louis, Washington, D. C., and elsewhere indicate that a more rigid requirement would be a worthwhile safeguard against damage to persons and property. The following requirement is believed to be an adequate safeguard as protection against wind damage:

**MAXIMUM HEIGHT OF PARAPET WALLS**

**For an assumed maximum wind velocity of 60 miles per hour**

| Wall thickness, inches | 8 | 12 | 16 | 20 |
|---|---|---|---|---|
| Maximum height, feet | 2.0 | 3.0 | 6.0 | 9.0 |

For other velocities multiply the height values by $\frac{3600}{V^2}$, where V = assumed wind velocity.

For resistance to earthquakes of the intensity of 1/10 g, the height should not be greater than twice the thickness.

This applies to parapet walls built of plain brick masonry. The introduction of horizontal continuous steel reinforcement near the bottom and top is recommended as a means of keeping parapet walls straight and preventing cracks or opened joints, which often are the source of damage to the building. R-B-M construction may be used to advantage to provide stronger and lighter construction and greater safety.

The flashing and dampproofing of parapet walls is of great importance and is fully covered in Chapter 8. Briefly, it consists of through flashing at the base and top of the parapet wall. The back of the wall should never be coated or sealed, but left exposed to permit rapid drying.

The quality of brick used should also be given careful consideration. The back of the parapet wall should be built of the same well-burned grade of brick as is used for facing. The practice of using for this purpose anything that may be left on the job should be prohibited, as it is frequently the cause of rapid deterioration and failure of the wall and damage to the building resulting in costly repairs.

Careful workmanship on parapet walls is most important. All joints should be filled, coping well laid and all called-for flashings properly built in. Outer joints should be concave, pointed with a round jointer and coping joints made tight, with coping laid on a full mortar bed. Proper workmanship on parapets and copings will make them watertight, strong and fire-resistive — all important qualities.

## 512. COPINGS

The design of copings is based principally on consideration of resistance to water penetration. It is important to provide for expansion and contraction due to changes in temperature and especially to changes in moisture content. This latter consideration applies particularly to copings of cement products, such as concrete or concrete blocks. Expansion and contraction of concrete, due to variations in moisture content, is much greater than similar movement in clay products and this differential movement between such coping and the brick wall beneath is sometimes the cause of rupture of the joints in the brickwork, which allows water to penetrate the walls. For this reason alone, brick wall copings should preferably be of brick or other burned clay material, such as special coping tiles.

## 513. PILASTERS AND BUTTRESSES

The common method of giving additional stiffness and stability to otherwise unsupported walls is by the use of pilasters or buttresses, built into and bonded with either the exterior or interior wall face. A mathematical calculation of the gravity reaction of such constructions is too involved for practical use and hence not material here. Graphical analyses are less laborious and sufficiently accurate for the purpose, since allowable design stresses involve a factor of safety sufficient to cover small errors of calculation.

A pilaster or buttress may fail by wholly or partially overturning, or by sliding of one part on an adjoining part at the joint between them. Both sources of failure should be examined in the design.

Sliding will occur when $H = fw$, in which H = the sum of the horizontal components of the forces acting above the joint, f is the coefficient of friction, and w is the weight of the portion above the joint.

The following coefficients of friction (equal to tangent of the angle of repose) may be used in design calculations:

| | |
|---|---|
| Brick on brick | 0.65 |
| Brick on building stones | 0.65 |
| Brick on moist clay | 0.33 |
| Brick on sand | 0.40 |
| Brick on gravel | 0.60 |

Conditions of stability are satisfied when the resultant reaction through all sections and the base of the pilaster or buttress passes through the middle third of the section or base.

## 514. CORBELLED SUPPORTS

When brickwork is corbelled out to provide support for other structural members, the projecting brick and each projecting course acts as a cantilever beam. The empirical

rule, which has proven safe in practice, is that each succeeding course shall not project more than two inches beyond the course below; and that the maximum horizontal projection beyond the face of the wall shall be not more than one-half the wall thickness.

Such a method of supporting other structural parts constitutes an eccentric load on the wall and the resultant of this load, especially if excessive, should be examined for its effect on wall stability.

Where concentrated loads are supported by corbelled brickwork, the allowable working stress should not be exceeded. If the same safety factor is to be preserved, a decrease of allowable loading in the corbelled section is warranted; 25% decrease is recommended.

## 515. CHASES AND RECESSES

Provisions for the construction of chases in walls or piers are based on arbitrary rules derived from experience. Building code provisions usually govern such construction. In general, chases should not be built within the required sectional area of piers, nor in 8-inch walls. They should not be deeper than one-third the wall thickness. No horizontal chase, nor the horizontal projection of a diagonal chase, should exceed four feet in length, unless special provision for additional support of the masonry above is made.

Recesses for stairways, elevators, and for alcoves and similar architectural features, should not be of such dimensions as to impair wall strength or stability. In general their depth should still leave 8 inches of wall thickness at the back. When more than eight feet in width or height, suitable support for the brickwork above should be provided by reinforcement in the brickwork, arches or other structural members.

## 516. SILLS

Brick or stone sills should be used in a brick building. Wood or cement sills are not well adapted for this purpose, nor are they sightly or permanent. Stone sills should be of such thickness that they line with the courses of brick, and they should preferably have a square lug at each end. The top surface should be formed with a slope and the sill should project one inch over the brick, with an undercut drip.

Brick sills are the least expensive. The brick are laid on edge and sloped outward 2″ to the foot for windows and ⅜″ for doors, with the bottom edge projecting about an inch to form a drip. The bottom edge of the brick may be accurately lined by means of a wood strip fastened to the face of the wall. Cement or cement-lime mortar should be used, a smooth bed spread, and the entire side of the brick should be covered with mortar before the brick is pressed into place. It is important that the joints be completely filled, pressed tight and pointed. Sills should be finished before the frames are set as it is more difficult to secure satisfactory results if the sills are placed later.

## 517. FRAMES

Frames of any type may be easily built into brick walls, whether solid or hollow type. To prevent infiltration, it is important that all frames be built with a windstop on the back so there is no straight joint through the wall. These stops assist in holding the frames in place, but door frames should also have heavy metal anchors fastened to the jambs and built into the masonry in preference to wood nailing blocks.

Frames are usually set by other workmen, but the mason should see that they are well bedded in mortar at the sill and rigidly braced in position in a manner that will not interfere with the work of the bricklayers. Horizontal braces across frames may be advantageous in keeping the jambs parallel, but a good bricklayer can build the brickwork tight against the frames without crowding or distorting them.

When R-B-M lintels are used, vertical braces should be placed in the frames, tightly wedged, to support the load and prevent deflection of the head until the masonry has set. A temporary soffit form will also be required for the support of the brick lintel projecting over the outside and inside edges of the frame.

### 518. SOLID WALLS

The standard size of brick — 2¼″ x 3¾″ x 8″ — is such that nominal wall thicknesses are conveniently given in multiples of four inches: Where actual dimensions are important, they should be carefully determined on the basis of brick size and joint thickness.

Non-load-bearing partition walls may be built of a single wythe of brick in running bond, laid edgewise or flat to form walls 2¼″ and 3¾″ thick, respectively. They may be plastered directly on one or both sides and offer the advantages of strength, fireproofness and high resistance to sound transmission.

Exterior and load-bearing walls may be of any thickness necessary to meet requirements of their particular use.

### 519. HOLLOW WALLS OF BRICK

Hollow walls of brick provide an economical and satisfactory type of construction possessing ample strength for most requirements. They are relatively light in weight, are highly fire resistive, dampproof if properly constructed, present the same exterior appearance as solid brickwork, can be used as backing for any facing material, and are low in cost. Hollow walls may be of several types, constructed according to the following details and descriptions. Under the name "Ideal Walls" are grouped Rolok-bak, All-rolok common bond and All-rolok Flemish bond.

### 520. ROLOK-BAK WALLS

The Rolok-bak wall is a general utility wall and may be employed for exposed or unexposed walls and for basement construction. The exterior wythe, or 4″ thickness, is laid with the brick flat in the exposed face, therefore, has the appearance of brickwork laid in the usual way and may be faced in any bond or pattern. The backing wythes are laid with brick on edge and bond is obtained by means of header courses at regular intervals.

The nominal thicknesses of Rolok-bak walls may be 8″ or in multiples of additional 4″ thicknesses. In the 12″ thickness, there are two types of construction — standard and heavy-duty.

The standard 12″ wall is designed for bearing walls of buildings where 12″ walls are required and floor loads are moderate, such as apartments, hospitals, clubs, offices, etc. In this wall, the wythes are bonded every seventh course with brick laid in basket weave.

The heavy duty 12″ wall is designed for the support of heavy floor loads and differs from the standard wall in the manner of bonding. The fourth course of the inner wythes (on edge) becomes a continuous rowlock header course bonding the two inner wythes. The next course is laid flat to form a continuous header course bonding the outer wythe and backed by a stretcher course on the inner face.

The quantities of brick required per square foot for the construction of Rolok-bak walls are as follows:

8″ thick. . . . . 7.0 exposed brick and 3.5 backing brick
12″ standard. . . . 6.6 exposed brick and 8.4 backing brick
12″ heavy duty. . 7.0 exposed brick and 8.8 backing brick

## 521. ALL-ROLOK COMMON BOND

All-rolok walls may be used for exposed and unexposed bearing or non-bearing walls. They are low in cost and light in weight and afford an opportunity to effect savings in the amount of steel required for the support of walls in skeleton frame construction.

The type of wall may be 8″ in thickness or greater, increasing by multiples of 4″. The wall is constructed by laying up all wythes with brick on edge, with two courses of stretchers on edge alternating with one continuous header bond course laid flat. The bond course is laid in basket weave pattern in the 12″ wall, and the center wythe is placed not at the center of the wall, but at the end of the headers which show on the exterior face of the wall.

The brick required per square foot for this type of wall are as follows:

8″ thick. . . . 9 brick, of which 6 are exposed on the outside
12″ thick. . . 13.5 brick, of which 6 are exposed on the outside

## 522. ALL-ROLOK FLEMISH BOND

This wall, because of its appearance, is intended primarily for exposed walls. It is very strong and consequently may be used for interior and basement walls. It is constructed entirely of brick on edge in Flemish for the outside 8″ thickness.

The 8″ wall is built of alternate stretchers and headers, backing up at every course. Headers in each course are placed over the center of the stretchers of the course below and should be carefully lined up vertically for the height of the wall.

For thicker walls, a wythe of stretchers is added for each additional 4″ thickness. The inner wythe is built three courses high, on which is laid a continuous rowlock bond course opposite which, on the exterior wythe, is laid a course of stretchers and bats to preserve the Flemish bond.

Brick required per square foot for this wall are:

8″ thick. . 12 brick, of which 6 are exposed on the outside
12″ thick. . 13.75 brick, of which 6 are exposed on the outside

## 523. CAVITY (Dual Masonry) WALLS

The cavity wall is frequently called the barrier wall or dual masonry wall.

This design of wall is becoming more and more popular in this country, because the continuous air space or cavity forms a barrier to the penetration of moisture and heat. It is not a new type, but has been in use for many years, particularly in England and Australia. It is suitable for use for exterior or interior bearing or non-bearing walls above grade and the 10″ (over all thickness) wall has ample strength for the support of loads usually present in buildings for residential or office class of occupancy up to two stories in height. Under heavier loads the inner wythe should be increased to 8″ in thickness.

The Cavity Wall is constructed of two wythes of brick laid flat with an air space usually 2″ wide, although this dimension may be varied. Masonry bond is not used, but the wythes are connected by metal ties placed in every fifth mortar bed and not more than 3″ apart horizontally. These ties may be made of ¼″ diameter reinforcing rods bent into a square "Z", $\frac{3}{16}$″ diameter rust-proofed rods, bent into a "U", or welded metal mesh with equivalent area of steel. If the wall is non-load-bearing, the inner wythe may be of brick laid up on edge.

The air space or cavity is provided with weep holes, formed by omitting mortar from vertical exterior joints at the bottom of the air space to permit the escape of any moisture that might accumulate. The bottom of the air space must be at least one course above the finished grade to prevent the entrance of moisture, and it should also be at least one course below the damp check. The cavity must be kept entirely free from mortar

droppings or debris. This may be accomplished by the use of strips laid on the ties and carefully lifted out before the next row of ties is placed.

Flashing is necessary over all openings, extending at least 6″ beyond the jambs. This may be of heavy waterproofed membrane or light metal such as 2 oz. copper.

Framing members are supported on the inner wythe and a row of ties is required in the bed joint immediately below the bearing course. Wood members should be fire cut and the ends protected with wood preservative. At the top of the wall, the air space is closed by a header course on which may be placed a wall plate to receive the roof framing.

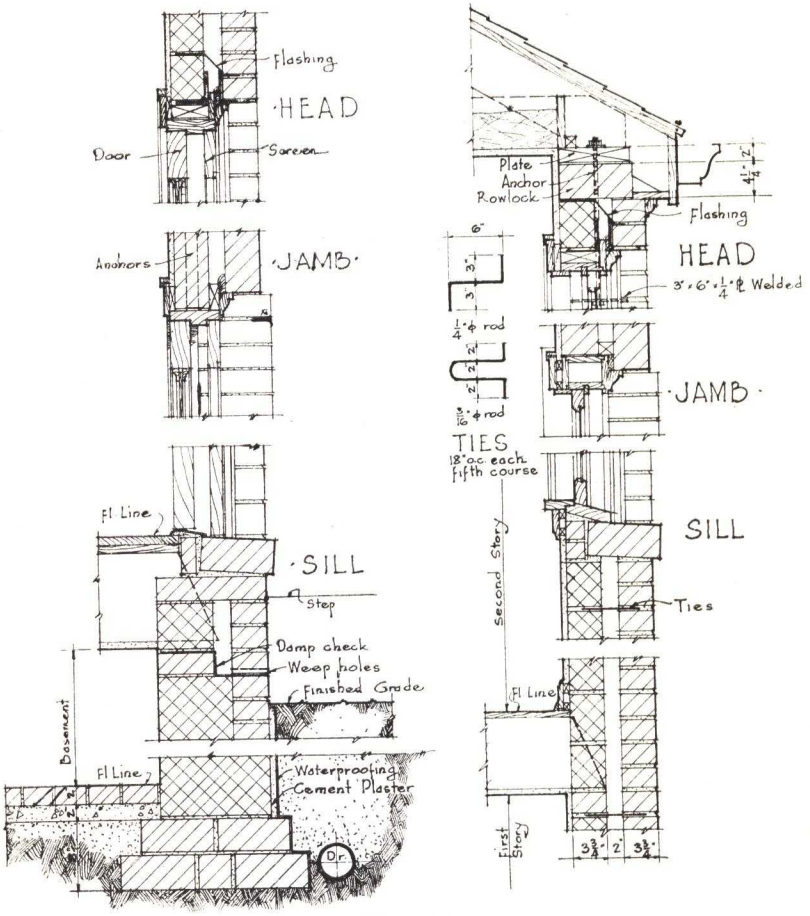



*Figure 19*
*Detail Sections of Dual Masonry (Cavity) Wall.*

If the air space is to be considered as an insulating medium, the top should be closed tight. In warm climates it may be found advantageous to ventilate the cavity, which may be accomplished by increasing the openings at the bottom and providing openings to the exterior at the top, protected against weather.

The number of brick required in the 9½″ or 10″ wall is 12.4 per square foot, 6.2 for each wythe, and the number of ties should not be less than one to three square feet.

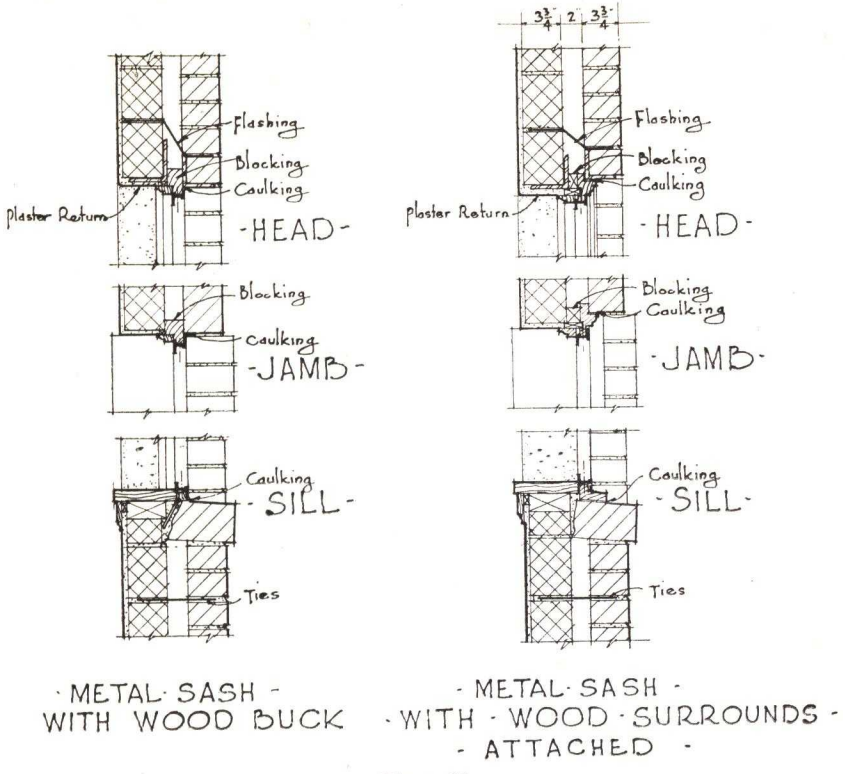



*Figure 20*

## 524. OTHER SPECIAL TYPES OF WALL

Various special shapes have been developed for use in the construction of hollow walls with brick headers. The purpose of these shapes in most types is to eliminate the use of through headers. Outstanding of the types of construction using special shapes are the Farren-Wall and Cain Air-flow Wall.

In both of these constructions, the cavity in the wall is utilized as part of a hot-air heating system. Cold air return ducts are placed in these cavities, extending from registers at the base of floors down through a closure course which seals the cavity in the basement wall below the frost line. By means of hollow tile cells, or other ducts in the basement floor, this lower cavity in the basement wall is connected to the air inlet or blower on the furnace.

When operating as a heating system the air returned to the lower cavity tends to warm the inside of the basement wall and thereby reduces the possibility of condensation on the inside of said wall.

In the summer time, cool air from the basement cavity is circulated into the upstairs rooms to provide a simple form of air conditioning.

## 525. ECONOMY WALLS

The Economy Wall is a 4″ wall built of brick laid flat and stiffened at suitable intervals by pilasters projecting 4″ from the wall. It is designed primarily for one-story

cottages, garages, filling stations or similar buildings, but may also be used for two-story buildings where building codes permit. The exterior appearance of the wall is practically the same as that of any brick wall laid in common bond, except for the headers at the pilaster points. Other bond patterns may be used by simply spacing the pilasters at the proper distances to fit the location of headers in the pattern.

Pilaster spacing may vary from two stretchers between 4″ wide pilasters to four and one-half stretchers between 8″ wide pilasters. Corners and jambs of openings must always be formed with pilasters preferably 8″ wide. Pilasters may also be built on both sides of the wall, forming a pier and panel wall, and if, for design purposes, the piers are spaced at wide intervals the panels may be stiffened at intermediate points with pilasters on the back of the wall.

For greater lateral strength and permitting wider spacing of pilasters, these walls may be reinforced by embedding continuous ¼″ bars or 3″ strips of expanded metal or welded mesh in every fourth or fifth horizontal mortar bed. Vertical reinforcing bars may also be placed in pilasters that are at least 8″ square.

The details show the method of constructing Economy Walls. Courses should be carefully laid out in advance to determine the proper location of pilasters with relation to headers, corners and openings. The brick should be laid on smooth mortar beds, not furrowed, and all vertical joints completely filled to produce maximum strength and resistance to moisture penetration. As a further precaution against dampness, the backs of the panels may be parged (plastered) which also reduces the heat transmission through the wall.

Supports for floor or roof members, sills and lintels are built 8″ thick with bars in the bottom mortar bed to form R-B-M beams supported on the pilasters. The correct depths and bar sizes should be determined for the loads and spans in accordance with design data given in Chapter 10. Construction of these members requires only the erection of temporary soffit forms properly shored in place.

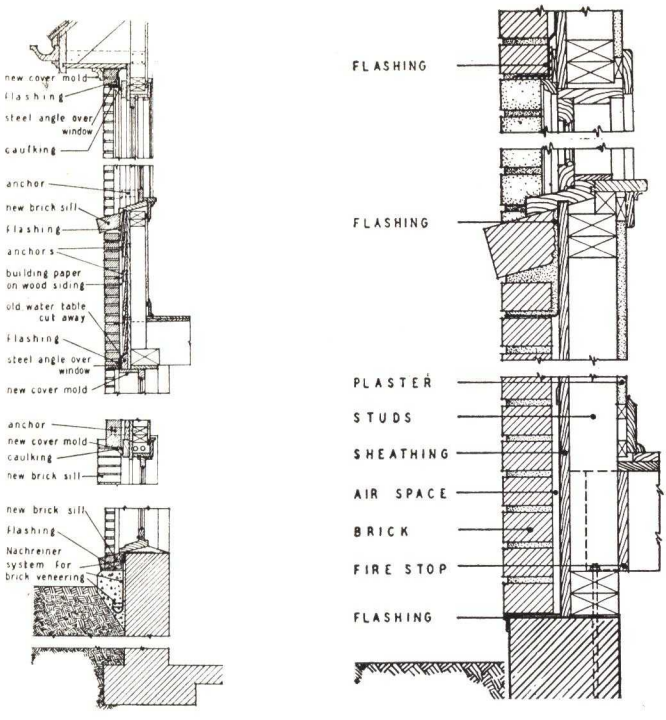
A flashing strip should be provided above openings if the wall is only 4″ thick at this location. The purpose is to divert any condensation which might accumulate and keep it from reaching the frame. This strip is shown in the detail and should extend at least 6″ beyond the jamb pilasters and have the upper edge bent out at least ½″ from the brick or mortar.

Interior finish may be applied to Economy Walls in several ways. Wood furring may be erected, 1″ thick where the strips occur over pilasters and 2″ thick elsewhere, to receive lath and plaster or wall board. Expanded metal or woven wire lath may be applied and fastened directly to the pilasters or to metal furring strips on the pilasters, held by means of wire ties built into the mortar beds. The latter is preferable since it eliminates combustible materials entirely from the wall construction. The number of brick required per square foot is 6.2 for the facing plus the number required for the pilasters, which must be estimated according to the size and spacing of the pilasters.

## 526. BRICK VENEER

The use of brick veneer as a facing material only, without utilizing its load-bearing properties, has found applications principally to dwellings in many parts of the country. Such veneering is usually applied over wood framing and sheathing of both old and new houses. It lacks many of the desired qualities of solid brick wall construction, brick and tile or even hollow walls of brick, but may be advantageously employed to rehabilitate an old wood or stucco surface.

For new construction, the cost of a brick veneered wall is about the same as the cost of a solid or hollow 8″ brick wall.




*Brick veneer on old frame construction.*

*Brick veneer on new construction.*

*Figure 21*
*Typical details of Brick Veneer on frame construction.*

In appearance, a brick veneer wall may have most any distinctive character and colorful beauty. The brick may be laid in any bond or pattern formed by the use of half brick as headers.

Structural stability is obtained by (1) sufficiently strong and well braced frame backing; (2) ample support, and anchorage of the veneer to the backing and (3) good construction of the brickwork.

Veneered walls resist exterior fire exposures better than frame, but have about the same internal resistance.

In new construction, the foundation walls should extend 5″ outside the face of the wood sheathing to receive the brick veneer. On old buildings the veneer should be started on the projecting portion of the footing or on a steel shelf angle bolted to the foundation wall, but never on an angle fastened to wood sill or framing members.

The wood walls should be covered with waterproof building paper and the veneer anchored in place with non-corrodible metal ties spaced not more than 16½″ apart vertically and 24″ horizontally.

Steel angles are used over openings and where the veneer extends above roofs firmly lag bolted to the framing.

Reinforced brick veneer is formed by applying to the sheathing a fibrous backed welded wire mesh of heavy gauge. The space behind the brick is then slushed full of mortar and the wall ties are attached to the fabric. The number of brick required is 6.2 per square foot.

## 527. BEARING OF FRAMING MEMBERS

Where plates, joists or other framing members bear on masonry, the brickwork should be finished to the required level with one or more header courses to bond the full thickness of the wall or pier. In hollow brick walls or walls of hollow units the bearings should consist of at least two courses of solid brickwork with header bond, except in the Cavity Wall where intermediate floors are supported on the inner wythe alone.

## 528. ANCHORAGE

The need for positive anchorage of walls to floors and roofs is readily apparent when it is considered that horizontal forces from wind or earthquakes may act both inward and outward on the walls. The friction between framing members and wall is not sufficient anchorage for even very moderate wind pressures.

Wood members are usually fastened at the ends by means of metal anchors fastened to the member near the bottom and having hooked or pier ends embedded in the masonry. At the sides, they are fastened by metal anchors fastened across the top of two or three members, with hooked or pin ends in the masonry. Wall plates are fastened by bolts with steel plates at the bottom and built into the masonry. Such anchors should be spaced four to six feet apart. The end studs of wood partitions against masonry walls should have at least three bolts in their height, built into the wall.

Steel members may be anchored by pins through the embedded ends, or other types of attached anchors with hooked or pier ends in the masonry.

Reinforced brick masonry or concrete has a natural bond with masonry, which may be increased by suitable bending of the reinforcing bars.

## 529. FURRING

The purposes and needs for furring are discussed in Chapter 8, so only the types and methods of application will be covered here.

Wood furring consists of wood strips 1″ x 2″, 2″ x 2″ or larger, applied to the face of the wall or over recesses. The spacing is governed by the dimensions of the lath or other covering material and the strips are fastened by nailing into blocks or plugs in the wall.

Metal furring consists of standard light steel channels, which are usually fastened by means of tie wires built into the mortar joints, or by patented clips made for this purpose.

Masonry furring may be of brick laid on edge independent of the structural wall and requiring no tie to the wall. Burned clay furring tile may also be used, this being tied to the wall by means of nails driven into the wall joints and bedded in the tile joints.

The type of furring material to be used should be selected on the basis of its permanence, fire safety and efficiency in producing the results desired.

## 530. NAILING PLUGS

The usual method of fastening wood to masonry is by means of nailing strips of blocks. Wood lath may be built into the mortar beds or wood blocks cut to brick size may be built into the masonry. The objection to this method is that the wood replaces mortar and brick and therefore reduces the strength of the wall. Shrinkage tends to loosen the wood and destroy its effectiveness in holding the finish properly in place.

Metal nailing plugs are recommended as providing better construction. These plugs are built into the mortar joints without weakening the structure and remain tight. On frames and rough bucks, flat metal anchors with hooked ends built into the masonry are preferable to nailing blocks.

# CHAPTER 6

# CHIMNEYS, FIREPLACES AND STACKS

## 601. SUITABILITY OF MATERIALS

Solid brickwork is the safest and most satisfactory material to use for the construction of fireplaces, chimneys and stacks. If a chimney fire occurs, considerable heat may be developed in the chimney and the safety of the building will then depend upon the integrity of the flue wall. Cracked or leaky flue walls are not only a fire hazard but may permit the passage of smoke and gases into adjacent rooms and may also destroy the efficiency of the flue.

The size of brick is such that they work perfectly with the standard flue linings and permit the thorough bonding of all parts of the fireplace and chimney construction with the minimum of cutting.

From the standpoint of appearance, the natural textures and variety of shadings make brick a particularly adaptable material in the hands of the architect in carrying out the design of the fireplace.

## 602. CHIMNEY DESIGN

A chimney is an integral piece of masonry from its foundation to top and may contain one or more flues and fireplaces. A flue is a space or passage in a chimney provided for the purpose of conducting smoke, fumes or gases to the outside air. A fireplace is an opening built into a chimney to contain an open fire and so designed that it will radiate heat into the room and deliver all smoke into its flue.

The effective operation or draft of a flue is influenced by its size, height and freedom from leaks and friction. Size and height of flues will be taken up later. The minimum height of a chimney is established by its relation to nearby obstructions, and therefore, the top of a chimney should extend at least 3 feet above an adjoining flat roof or parapet wall and at least 2 feet above the highest ridge of a pitched roof, unless otherwise required by the local building ordinance. This is to prevent down drafts resulting from air currents curling over walls and roofs. Nearby trees will produce the same results and consideration should be given to this condition.

Chimneys located entirely inside a building are more efficient because they do not become chilled by outside air but retain their heat and consequently give a better draft. For this reason, chimneys built in exterior walls may be made more efficient by increasing the thickness of the flue walls to retain as much heat as possible. No change in size or shape of the chimney should be made within 6″ above or below the roof construction where they intersect.

Flues should be built as straight as possible, since offsets increase friction, collect soot and may form a fire hazard. If offsets are necessary, their slope would not exceed an angle of 30° from the vertical and the full area of the flue should be maintained throughout. For the safest and most efficient operation, flues should be smooth and contain as few joints as possible. Fireclay flue lining should be used and is required in many building ordinances. It produces smooth inside surfaces, few joints, is highly fire resistive and simplifies the construction of the chimney.

Not more than two lined flues should be built in the same chimney space and additional flues should be separated by a smoketight wythe of brick, 4″ thick. When two linings are set together, their joints must be staggered at least 7″ apart. Each flue serving a heating boiler, furnace or fireplace must be separated or surrounded by at least 4″ of

brick masonry. The best chimney design provides for the separation or surrounding of every flue with 4″ of brick, which provides more satisfactory draft, greater fire safety and increased stability of construction.

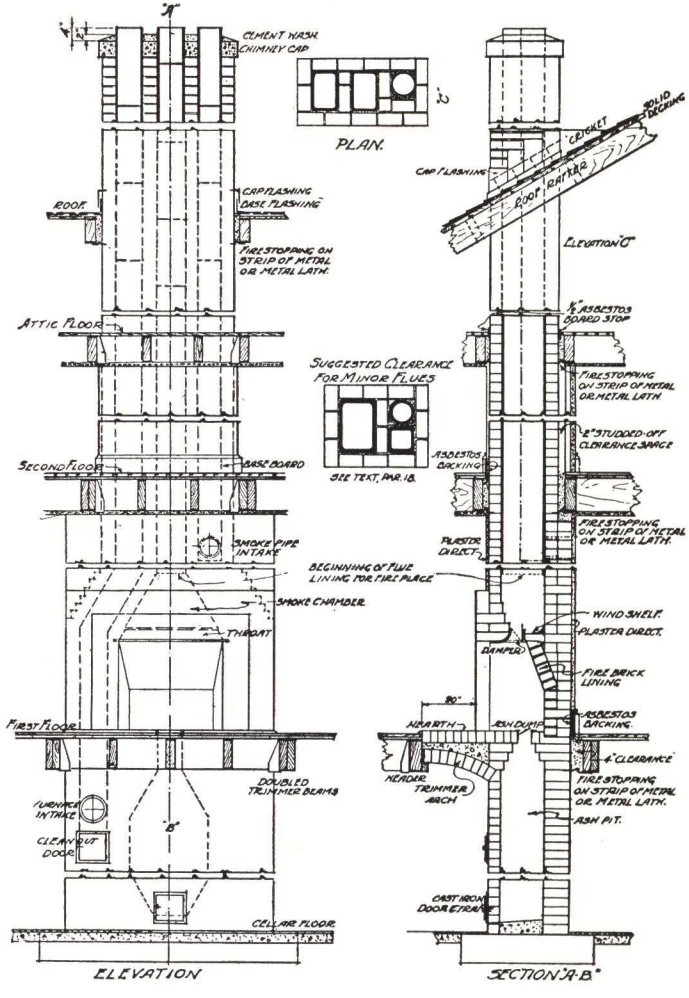



*Figure 22*

*Elevation, plan and section of a chimney.*

## 603. CHIMNEY SUPPORT

Because of the greater mass and height of a chimney, it is quite probable that the load on its foundation will be greater than that on adjoining foundations. For this reason, it is important that consideration be given to the proper design of the footings so the resultant unit load on the ground will be the same for all footings in the building. This will prevent uneven settlement and the possibility of damage to the finished structure.

Chimneys must be supported on solid masonry foundations or self-supporting fireproof construction and never on wooden construction. When supported by a solid masonry wall not less than 12″ in thickness, a chimney may be corbeled out not more than 6″ and the corbeling should not project more than 1″ in each course.

### 604. CHIMNEY FOOTINGS

The construction of footings is described in Chapter 4 and applies as well to chimney footings, except for one condition; the projection of a footing is designed to resist shear. The hollow space at the bottom of a chimney is sometimes wider than twice the normal footing projection. If this is the case, the footings should be extended and built as a unit and the center part of the footing pad should be reinforced near the top; or the wall thicknesses may be increased by corbeling out the bottom courses of brick to the required width.

### 605. CHIMNEY BASE

The base of a chimney containing only flues may be built with the outside walls 3¾″ thick. If a fireplace is included in the chimney, the walls at the ends and under the back of the fireplace should be at least 8″ thick.

The ash pit should have a tight fitting cast-iron clean-out door with frame of the same material, securely anchored in place and set approximately 16″ above the floor, or, if there is no basement, the cleanout may be located one or two courses above the finished grade line if accessible from the exterior of the building. Flue lining is not required below a point 8″ under the lowest smoke pipe thimble or flue ring. At the bottom of each flue, other than fireplace flues, a soot pocket and cleanout should be formed. The cleanout should consist of a tight fitting cast-iron door and frame built solidly into the masonry. The top of the door frame should be not more than one brick course below the bottom of the first section of flue lining. The soot pocket should extend to one course below the bottom of the door and should be plastered smooth on the inside. Each flue must be built as a separate unit entirely free from any connection with other flues or openings.

### 606. FLUE SIZES

Smoke travels through a flue with a circular swirling motion, and therefore, a round flue is considered the most efficient. For this same reason, the full area of a rectangular flue is not effective, the corners being dead areas. Table 23 gives the effective areas for standard sizes of flue lining, those for round linings being the full area and those for rectangular linings being determined by the formula, $E = \frac{\pi B^2}{4} + B\,(A\text{-}B)$ in which E equals the effective area and A and B equal the inside length and width respectively. The effective area of an unlined round flue is based upon a diameter 2″ less than the actual diameter and for an unlined rectangular flue, the effective area is computed from the formula $E = \frac{\pi\,(B\text{-}2)^2}{4} + (A\text{-}B)\,(B\text{-}2)$ in which the nomenclature is the same as in the previous formula.

The size and construction of flues for oil or gas fired heating units should be the same as required for the same units burning solid fuel. Vent flues for domestic gas appliances, having relatively small gas consumption should have an effective area of not less than 10 square inches. When two appliances are to be connected to one flue, the capacity of the flue should be taken as 60% of the value given in Table 23. For three appliances, use 40% and for four appliances, use 35%.

The required flue area and height is available from the manufacturer of the unit or appliance to be used, and these data should be the basis for the proper design of each flue. Fireplace flues should have an effective area equal to 1/10 of the area of the fireplace opening if the chimney height is less than 30 feet, and 1/12 if the chimney height is more than 30 feet.

## TABLE NO. 23

## TABLE OF FLUE SIZES AND CHIMNEY HEIGHTS

| Warm Air Furnace Capacity Sq. In. of Leader Pipe | Steam Boiler Capacity Sq. Ft. of Radiation | Hot Water Heater Capacity Sq. Ft. of Radiation | Rectangular Flue | | | | Round Flue | | Height in Ft. from Grate |
|---|---|---|---|---|---|---|---|---|---|
| | | | Nominal Dimensions of Fire Clay Lining Inches | Actual Inside Dimensions of Fire Clay Lining Inches | Actual Area Square Inches | Effective Area Square Inches | Inside Diam. of Lining Inches | Actual and Effective Area Square Inches | |
| 790 | 590 | 973 | 8½x13 | 7 x11½ | 81 | 70 | .. | ... | 35 |
| 1,000 | 690 | 1,140 | ..... | ..... | ... | ... | 10 | 79 | .. |
| .... | 900 | 1,490 | 13 x13 | 11¼x11¼ | 127 | 99 | .. | ... | .. |
| .... | 900 | 1,490 | 8½x18 | 6¾x16¼ | 110 | 100 | .. | ... | .. |
| .... | 1,100 | 1,820 | ..... | ..... | ... | ... | 12 | 113 | 40 |
| .... | 1,700 | 2,800 | 13 x18 | 11¼x16¼ | 183 | 156 | .. | ... | .. |
| .... | 1,940 | 3,200 | ..... | ..... | ... | ... | 15 | 177 | .. |
| .... | 2,130 | 3,520 | 18 x18 | 15¾x15¾ | 248 | 195 | .. | ... | .. |
| .... | 2,480 | 4,090 | 20 x20 | 17¼x17¼ | 298 | 234 | .. | ... | 45 |
| .... | 3,150 | 5,200 | ..... | ..... | ... | ... | 18 | 254 | 50 |
| .... | 4,300 | 7,100 | ..... | ..... | ... | ... | 20 | 314 | .. |
| .... | 5,000 | 8,250 | 24 x24 | 21 x21 | 441 | 346 | .. | ... | 55 |
| .... | 4,600 | 7,590 | ..... | 20 x24 | 480 | 326 | .. | ... | .. |
| .... | 5,570 | 9,190 | ..... | 24 x24* | 576 | 452 | .. | ... | 60 |
| .... | 5,580 | 9,200 | ..... | See Note | ... | ... | 22 | 380 | .. |
| .... | 6,980 | 11,500 | ..... | ..... | ... | ... | 24 | 452 | 65 |
| .... | 7,270 | 12,000 | ..... | 24 x28 | 672 | 468 | .. | ... | .. |
| .... | 8,700 | 14,400 | ..... | 28 x28 | 784 | 531 | .. | ... | .. |
| .... | 9,380 | 15,500 | ..... | ..... | ... | ... | 27 | 573 | .. |
| .... | 10,150 | 16,750 | ..... | 30 x30 | 900 | 616 | .. | ... | .. |
| .... | 10,470 | 17,250 | ..... | 28 x32 | 896 | 635 | .. | ... | .. |
| .... | 11,800 | 19,500 | ..... | ..... | ... | ... | 30 | 707 | 70 |
| .... | 14,700 | 24,300 | ..... | ..... | ... | ... | 33 | 855 | .. |
| .... | 17,900 | 29,500 | ..... | ..... | ... | ... | 36 | 1,018 | .. |

*Note: Dimensions below are larger than those in which rectangular fire clay flue linings are commercially available and hence are for unlined rectangular flues — requiring thicker walls than when lined.

Flue heights and sizes are based upon an altitude not over 500 feet above sea level. For each one thousand feet above sea level the height of the flue for a given appliance

should be increased 10% above the figures in the table and the capacities in the table should be reduced 5%.

Flue sizes are based on run-of-mine soft coal but are to be used without reduction for any fuel. The smaller sizes of fuels require excessive draft and when the fuel to be used is screened smaller than the following, each installation requires especial consideration: Anthracite — nut, Bituminous — run-of-mine, Coke — nut.

The ratings given are based on smooth, lined, straight flues. If there is more than one offset or if any single offset is flatter than 60° with the horizontal, the rating should be reduced accordingly.

The flue heights and sizes are for ordinary up-draft boilers. For special types of boilers greater heights and area may be required.

Steam and water boiler capacities are the rated capacities of the manufacturer.

Since the effective area of a standard flue lining may seldom equal the required area, the next larger size should be used, but a few square inches less in area will not seriously affect the action of the flue.

### 607. SEPARATE FLUES

Irrespective of the kind of fuel to be used, a separate flue should be provided for each heating furnace or boiler, fireplace and incinerator, with only the one connection for the unit it serves. The same rule might well apply to all flues although for small units or low consumption gas appliances, more than one connection may be permissible. In all cases, the local building ordinance or the Code of the National Board of Fire Underwriters should govern.

### 608. FLUE WALLS

Chimneys may be built without flue lining provided the flue walls are at least 8″ thick and that the inner course is of refractory brick. Exposed joints, both inside and outside, should be struck smooth and the inside should **not** be plastered. Unlined flues are not smooth, and consequently soot accumulates, reducing the flue area and increasing the hazard of chimney fires. The use of flue lining provides a smooth interior surface, increases the efficiency of the flue and reduces fire hazard. The additional cost of flue lining is offset by the reduced thickness of the flue walls, which may be built of brick 3¾″ thick.

In all cases, flue walls must be built with the joints completely filled with mortar, including the joint against the lining. When round flue lining is used, particular care should be taken to insure complete filling of voids against the lining. Leaky joints may permit infiltration of air to reduce the draft, or the passage of flame to adjacent combustible materials, and be the cause of deterioration of the masonry where exposed above the roof.

Flues, serving gas-fired heating appliance, should always be lined with fireclay flue lining. The products of combustion from such a gas burner contain a large volume of water vapor and some acids, which condense on the inside walls of the flue because of the comparatively low temperature of the masonry. The penetration of this moisture and acid may cause efflorescence and discoloration, and the rapid deterioration of mortar joints. A good lining is necessary to stop such penetration. Another method of protecting the chimney is the application of an acid-resisting coating which can be applied with a spray and renewed every few years if necessary.

In large rectangular chimneys, as in apartment buildings, horizontal reinforcing bars are recommended. Such reinforcement, bent to form hoops and spaced every fourth or fifth course, serves to prevent vertical cracking of the flue walls due to temperature differences and changes.

## 609. FLUE LINING

Flue lining must withstand the action of fire and heat, smoke, gases and acids, and changes in temperature. It should, therefore, be made from fireclay or other suitable clay with a thickness of not less than $\frac{5}{8}$ inches.

Only whole sound units of lining should be used, and they must be set in mortar composed of one part Portland cement to three parts sand with the joints tight and completely filled and pointed smooth on the inside. Each section of flue lining should be set before the surrounding brickwork reaches the top of the flue lining section below and the brickwork then built around the flue.

Where offsets or bends are necessary, they should be formed by mitering equally the ends of both abutting sections of lining. This prevents reduction of the flue area, and it is important that the same effective area be maintained for the full height of the flue.

Flue lining can be cut by filling the section with damp sand, tamped solid and tapping a sharp chisel with a light hammer along the line where the cut is desired. No cutting should be done after the lining is built into the chimney, as it will probably be shattered and fall out of place. Round flue lining is not only more efficient but it remains cleaner. The corners of rectangular linings tend to collect soot which may accumulate to the extent of reducing the effective flue area, and, if this soot burns out, it presents the danger of fire from sparks falling on the roof.

## 610. CONNECTIONS

Standard flue lining sections are available with openings to receive smoke pipe connections, but if the opening is to be cut, it should be done in the manner described above and not after the chimney is built.

The smoke pipe connection should enter the side of the flue through a thimble or flue-ring preferably made of fireclay. Such rings are available with inside diameters of 6″, 7″, 8″, 10″ and 12″ and lengths of 4½″, 6″, 9″ and 12″. These rings should be built in as the work progresses and must be made airtight at all points. The metal smoke pipe should not enter farther than the inner face of the flue, and the joint around the pipe should be made airtight with boiler putty or asbestos cement. The top of the smokepipe should be not less than 18″ below the ceiling, protected by metal or plaster, and no wood or combustible material should be placed within 6″ of the pipe.

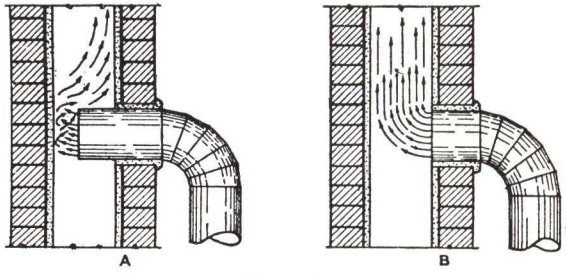



*Figure 23*
*Showing improper (A) and proper (B) connections.*

## 611. FIREPLACE HEARTH

The usual practice in hearth construction has been to form an arch of brick from the chimney base to the floor construction and filling over this arch to a level surface to receive the finished hearth.

Instead of such an arch, a reinforced brick masonry cantilevered slab is preferable and may be easily constructed. Ribbed metal lath serves as the form and is simply laid in a mortar bed on top of the brick walls of the chimney base and supported at the projecting end by a temporary strip on the floor framing. On this metal lath place a bed of cement mortar of sufficient depth to cover the ribs and follow at once with the setting of the brick on edge ½″ apart and with or without mortar. Fill the remainder of the joints full of cement mortar grout and set the reinforcing bars in place, pressing them down into their proper position approximately 1½″ below the surface.

The most simple method of constructing this slab is to let it cover the entire area of the chimney base for the width of the hearth. Cut out the lath for the ash chute and basement flues and set the brick to form the proper opening, closing the joints with mortar to hold the grout fill. Flue linings from below should be set to extend above the slab and the brick and grout placed solid around them.

## 612. SIZE OF FIREPLACE

Careful consideration should be given to the size of fireplace best suited to the room in which it is located, not only from the viewpoint of its appearance but of its operation as well. If it is too small, it may function properly but will not produce a sufficient amount of heat. If too large, a fire that would fill the combustion chamber would be entirely too hot for the room. Moreover, it would require a larger chimney and induce an abnormal infiltration of air through doors and windows to supply the needs for combustion and so waste fuel.

A room with 300 square feet of floor area is well served by a fireplace with an opening 30″ to 36″ wide. For larger rooms the width may be increased, but all other dimensions should be taken from Table 24 for the width opening selected.

## 613. ESSENTIALS OF A FIREPLACE

The building of a fireplace that will perform properly and satisfactorily is not a mysterious operation nor dependent upon luck. On the contrary, the essentials of correct design are but few and simple. The principal results to be achieved are:

1. Proper combustion of the fuel.
2. Delivery of all smoke and other products of combustion up the chimney.
3. Radiation of the maximum amount of heat into the room.
4. Simplicity and fire-safeness in construction.

Of these, 1 and 2 are closely related and depend mainly upon the shape and relative dimensions of the combustion chamber, the proper location of the fireplace throat, and its relation to the smoke shelf, and the ratio of the flue area to fireplace opening area. Three is dependent upon the shape and dimensions of the combustion chamber, and four is subject to the size and shape of the masonry units used and their ability to withstand high temperatures without warping, cracking or deterioration.

## 614. FIREPLACE OPENING

Fireplace openings should not be too high; for the usual width of opening, the height above the hearth is seldom more than 32 inches. The first two columns of Table 24 give the widths and corresponding heights of fireplace openings found to be the most satisfactory for appearance and efficient operation. These dimensions may be varied slightly to meet regular brick courses and joints, especially where the facing or trim is of brick.

## 615. COMBUSTION CHAMBER

The shape of the combustion chamber influences both the draft and the amount of heat radiated into the room. The correct splays and slopes are shown by the detail drawing, Figure 22, and the corresponding dimensions are given in Table 24. Again these dimensions might be varied slightly but the information given is based on years of experience and it is inadvisable to make any great changes. The slope of the back throws the flame forward and leads the gases with increasing velocity through the throat. It also, with the splay of the sides gives the maximum radiation of heat into the room. The combustion chamber should be lined with firebrick laid with thin joints of fireclay mortar. The back and end walls of the average size fireplace should be at least 8″ thick, and thicker for larger sizes, to support the chimney load above.

**TABLE 24**
**FIREPLACE DIMENSIONS**
**All Dimensions Below Are Given in Inches**

| Finished Fireplace Opening | | | | | | | Rough Brickwork | | | | Flue Sizes | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Width | Height | Depth | Width of Back | Vertical Back Wall | Sloped Back Wall | Throat | Width | Depth | Smoke Chamber | Slope of Smoke Chamber | Rectangular Flue Lining | Effective Area Sq. In. | Round Flue Lining Diameter | Effective Area Sq. In. |
| 24 | 28 | 16 | 11 | 14 | 18 | 8¾ | 37 | 20 | 24 | 14 | 8½x8½ | 45 | 10 | 79 |
| 28 | 28 | 16 | 15 | 14 | 18 | 8¾ | 42 | 20 | 25 | 14½ | 8½x13 | 70 | 10 | 79 |
| 30 | 30 | 16 | 17 | 14 | 20 | 8¾ | 42 | 20 | 25 | 14½ | 8½x13 | 70 | 10 | 79 |
| 34 | 30 | 16 | 21 | 14 | 20 | 8¾ | 46 | 20 | 28 | 16½ | 8½x13 | 70 | 12 | 113 |
| 36 | 30 | 16 | 23 | 14 | 20 | 8¾ | 46 | 20 | 28 | 16½ | 13x13 | 99 | 12 | 113 |
| 40 | 30 | 16 | 27 | 14 | 20 | 8¾ | 50 | 20 | 32 | 18½ | 13x13 | 99 | 12 | 113 |
| 42 | 30 | 16 | 29 | 14 | 20 | 8¾ | 54 | 20 | 35 | 20½ | 13x13 | 99 | 12 | 113 |
| 48 | 33 | 18 | 33 | 14 | 23 | 8¾ | 59 | 22 | 40 | 23 | 13x13 | 99 | 15 | 177 |
| 54 | 36 | 20 | 37 | 14 | 26 | 13 | 67 | 24 | 42 | 24½ | 13x18 | 156 | 15 | 177 |
| 60 | 39 | 22 | 42 | 14 | 29 | 13 | 71 | 26 | 45 | 26½ | 18x18 | 195 | 18 | 254 |
| 72 | 40 | 22 | 54 | 14 | 30 | 13 | 83 | 26 | 46 | 32½ | 18x18 | 195 | 18 | 254 |

## 616. THROAT

Because of its effect on the draft, the throat of the fireplace should be carefully designed. It should be not less than 4″ and preferably 8″ above the highest point of the top of the fireplace opening. The sloping back extends to the same height and forms the support for the back of the damper. A metal damper should be placed in the throat, extending the full width of the fireplace opening, preferably of a design in which the valve plate opens upward toward the back. Such a plate when open forms a barrier to stop down draft and deflect it upward into the ascending column of smoke. When the fireplace is not in use, the damper should be kept closed to stop drafts and heat loss from the room as well as to keep out dirt from the flue.

Metal dampers are available in several types, some forming in one piece the damper, throat and supporting lintel over the fireplace opening. This type has the advantage of producing a smooth throat passage and simplifying the masonry work.

### 617. SMOKE SHELF AND CHAMBER

The location of the throat established the position of the smoke shelf. This shelf should be directly under the bottom of the flue, extend the full width of the throat and must be constructed horizontal and not sloped, as its purpose is to stop the down draft.

The space above the shelf is the smoke chamber. The back wall of the chamber is built straight, the side walls are sloped uniformly to the *center* to meet the bottom of the flue lining, and the front wall above the throat is also sloped to the flue lining. The interior of the smoke chamber should be plastered with mortar troweled reasonably smooth.

Metal lining plates are available for the smoke chamber and are effective in giving the chamber its proper form and smooth surfaces, as well as simplifying the brick laying.

### 618. FIREPLACE FLUE

Relatively high velocities through the throat and flue are desirable and the velocity is affected by both the area of the flue and the height of the chimney. The proper sizes and heights of fireplace flues may be obtained from Table 23 or as recommended under "Flue Sizes."

The fireplace should have an independent flue entirely free from other openings or connections, and the first section of flue lining must start at the center line of the fireplace opening. This is important in obtaining a positive and uniform draft over the full width of the fireplace. The flue lining should be supported on at least three sides by a ledge of projecting brick, finishing flush with the inside of the lining.

### 619. FIREPLACE CONSTRUCTION METHODS

There are two methods of building a fireplace and chimney assembly. One is to complete the fireplace and chimney in one operation as the work progresses. The facing may be omitted until later, in which case suitable ties are built in for properly bonding the facing in place.

The other method is to form a rough opening for the later construction of the fireplace proper. In this case, the end and back walls are built 8″ thick, a steel lintel is set below the shelf level to carry the front wall above, the back and sides of the smoke chamber are formed and the flue built in. Care must be taken in building the fireplace to finish the work tight to the under side of the previously built chimney to prevent leaks and the possibility of fire hazard.

### 620. METAL FIREPLACES

Several types of steel plate fireplaces are on the market. These are made to combine in one unit the combustion chamber, damper and smoke chamber and need only to be set in place on the hearth and built into the chimney. They are hollow and are provided with openings for circulating the warmed air through ducts and registers.

### 621. DUTCH OVEN

A Dutch Oven may be desired in connection with a fireplace either for the purpose of completing the appearance of an early kitchen type fireplace or for actual use as an oven. In either case, it will have a certain value in actual use.

Included merely as a design feature, the oven will be fitted with a cast iron door and the space thus formed may be used for wood storage, and an open fire box below may be similarly used. The only precaution in this case is to see that the spaces are separated from the fireplace by a brick wall at least 8″ thick and well built with completely filled joints.

If intended for actual use in baking or roasting foods, the Dutch oven should be carefully designed. The entire interior of the oven should be lined with fire brick and the masonry thickness should be at least 8″, but greater thickness will increase the heat storing capacity of the oven. The ash drop should be a standard type cast iron unit and may lead either to the side of the fireplace or to an ash pit in the chimney base. The throat should be carefully formed and preferably fitted with a metal damper. A separate flue should be provided for the oven and for the oven sizes ordinarily used, an 8½ x 8½-inch flue lining will be found to be ample. The oven is pre-heated by fire or hot coals and when the brick are thoroughly heated, the coals and ashes are removed through the ash drop and the food placed in the oven.

## 622. OUTDOOR FIREPLACES

An outdoor fireplace may be intended for ornament and warmth, for cooking, or for both and recognized types have been developed for each class. No attempt will be made to describe the various types because of the great freedom enjoyed in designing these structures. Instead, a few points concerning their construction will be mentioned.

The foundation should, of course, extend below the frost line, although the hearth may be laid on a cinder bed and free of the main structure.

Consideration should be given to the fire resistance of the material used for the interior of the combustion chamber, particularly if local materials are to be used, remembering that fire brick is the most satisfactory for such exposure.

Chimneys should be lined with fireclay flue lining. In northern climates, it is advisable to carry up the corners of the chimney and cover the top with a stone slab to keep out rain and snow and prevent damage from freezing. Flue sizes should be increased because of their reduced height.

Fireplaces built for warmth should follow the same general lines as interior fireplaces, retaining the smoke shelf, but omitting the damper. The throat and flue should be larger and the side splays of the combustion chamber may be omitted.

## 623. FLASHING

At the intersection of chimney and roof, the connection should be made weathertight by means of flashing and counter flashing, preferably of copper or other rust-resisting metal, arranged to allow for any vertical or lateral movement between the chimney and roof.

## 624. CHIMNEY TOPS

The tops of chimneys above the roof offer unlimited possibilities for architectural treatment by the use of cut or ground brick in addition to the standard shape. In the more ornamental types, round flue lining may be found to be more advantageous and in this event the same round lining should be used for the full height of the flue.

Regardless of the architectural design, certain structural details should always be followed. The flue lining should project 4″ above the chimney cap or top course of brick and be surrounded with a wash of rich Portland cement mortar at least 2″ thick finished with a straight or concave slope to direct the air current upward at the top of the flue. This wash also serves to drain water from the top of the chimney and it is preferable to form the cap or top courses with sufficient projection to serve as a drip and keep the

walls dry and clean. The masonry flue walls and wythes should be well bonded either by carefully staggering the joints or by the introduction of non-corrosive metal ties.

Fireclay chimney tops or pots are available for use on new or old chimneys. Care should be exercised in selecting these units so that the effective area of the flue is not reduced. They are made to fit over the flue linings, one to each flue, and should be set in Portland cement mortar finished to form a wash as above described.

### 625. INCINERATORS

Because of the various types and sizes of incinerators and rubbish burners, reference should be made to the local ordinance or the Code of the National Board of Fire Underwriters for regulations governing the construction of combustion chambers and flues.

### 626. ADJACENT WOODWORK

Wood framing and furring members should never be placed closer than 2″ to the walls of a chimney and this distance should be increased to 4″ at the backs of fireplaces. The space between chimneys and floor members should be filled with an incombustible insulating material supported on strips of metal or metal lath buckled to form a flexible joint.

Plaster may be applied directly on the chimney walls or on metal lath and metal furring. If wood nailing blocks are required for trim, they may be set with asbestos paper or board at least ⅛″ thick between the wood and masonry and held with nails into metal nailing plugs built into the masonry.

### 627. MATERIALS

Brick chimneys should be built of brick of good sound quality and where exposed on the exterior, they should also meet requirements for weathering. The interior surfaces of unlined flues in residences should be faced with heat-resisting brick, having a softening point at 1922 degrees Fahrenheit or over. Such brick are generally available and in many localities, the regular run of brick will meet this requirement.

Fire brick used for lining flues and fireplaces should be laid in fireclay mortar.

Cement mortar should be used for all joints in fireclay flue lining and for all parts of chimneys where exposed to the weather. All other parts of the chimney construction may be laid up in cement-lime mortar. The proportions and mixing of mortars are described under Chapter 2.

### 628. QUANTITIES

The quantities of brick and mortar required for ordinary straight chimneys, both free standing and attached, are given in Table 72 in Appendix. For fireplaces, the quantities may be estimated by considering the fireplace as a solid wall from floor to floor, taking the quantities from Table 72 in Appendix and deducting the number of brick displaced by the combustion chamber and flues.

### 629. WORKMANSHIP

This subject has been covered in Chapter 3, but certain details are important in chimney construction and should be given particular attention. Brick should be laid on smooth mortar beds, not grooved, and the end joints completely filled. The space between brick and flue lining should be completely filled by slushing or grouting.

Flue lining should always be set in advance of the surrounding brickwork and the joints completely filled and pointed smooth on the inside. The bottom section of flue lining should be supported on at least three sides by brick courses projecting to the inside surface of the lining. To prevent the accumulation of mortar droppings inside the

flue, a tight-fitting bag of shavings should be drawn up by a rope as the flue is built. After completion, all flues should be thoroughly cleaned and left smooth on the inside.

### 630. SMOKE TEST

After the chimney has been completed and the mortar thoroughly hardened and before any appliances are connected, a smoke test should be made on each flue. This is done by building a smudge fire at the bottom of the flue, and, while the smoke is flowing freely from the flue, cover the top tightly. Escape of smoke into other flues or through the chimney walls indicates openings that must be made tight before the chimney is accepted. The test should be made by the mason contractor and observed by representatives of the Building Department and the Owner.

Repairing of leaks is usually difficult and expensive, and it is, therefore, much more satisfactory and economical to see that construction is properly executed as the work progresses.

### 631. CLEANING FLUES

Smoke passages and chimney flues should be kept clean. An accumulation of soot may cause a chimney fire with consequent danger of sparks igniting the roof or damage to the chimney itself which would permit passage of fire through the flue walls.

The most efficient method of cleaning a chimney flue is by means of a weighted brush or bundle of rags lowered and raised from the top.

### 632. REPAIRING

The tops of unlined chimneys may have to be rebuilt every few years due to the disintegrating effect of smoke and gases on the mortar. The chimney should be taken down to a point where the mortar joints are solid. The new top should be built with fireclay flue lining of the same size as the old flue, the construction being as above described.

### 633. DETACHED CHIMNEYS OR LARGE STACKS

Large chimneys or stacks for high heat appliances, large heating and industrial plants are usually detached structures independent of adjacent buildings. They contain a large single flue and connections for multiple units are made in the metal breeching leading to the chimney.

No attempt will be made to cover the theory of design of this type of chimney as these stacks present individual problems peculiar to the particular requirements. Such a chimney or stack should be designed by an engineer familiar with the requirements, both mechanical and structural, of the individual case.

Large chimneys need not be unsightly and their appearance may be greatly enhanced by a careful study of their architectural design. There are many examples of large chimneys which, because of their architectural treatment are actually attractive features of building groups and in most of these instances, the results were accomplished by the use of brick. The flexibility of brick as a masonry unit permits the maximum of freedom in design and simplicity of construction.

Most chimneys are built of brick, either standard size brick or special radial brick, which are usually perforated and molded to different sizes to fit the varying diameters of the chimney. These burned clay materials are highly resistant to the action of smoke and gases and are not affected by high and rapid changes of temperature.

Reinforced brick masonry permits the use of thinner shells or walls than are necessary in plain masonry chimneys or stacks. The foundation and footing may also be built of reinforced brick masonry construction.

# CHAPTER 7

# FLOORS, WALKS AND GARDEN STRUCTURES

## 701. FLOORS

The use of brick for floors may be divided into two classes — as paving and as a structural slab. This section will deal only with brick paving and methods of construction and finishing. The design and construction of structural slabs of reinforced brick masonry are fully explained in Chapter 10.

## 702. FLOOR BRICK QUALITY

Brick for interior floor paving or surfacing should be selected for the purpose it is to serve. Exposed surfaces should be smooth and straight to produce an even floor surface. For a residence basement floor, a well-burned brick hard enough to resist ordinary wear should be acceptable, but for an industrial floor subjected to heavy trucking and abrasion, an impervious floor brick should be used.

## 703. PREPARATION OF BASE

The soil under the floor or pavement must be firm and dry and all irregularities in the subgrade eliminated. The subgrade surface should be carefully graded to the proper elevation, either level or sloping to drain. Over this, place a bed of sand or granulated slag, not less than 2″ thick, rolled and tamped to the required grades.

If there is any doubt about the suitability of the subsoil, or if a heavier slab is required, lay a 4″ bed of 1:2½:5 concrete with a sand or granulated slag cushion. Greater strength may be obtained by the addition of mesh reinforcement.

## 704. LAYING BRICK

The brick may be laid flat or on edge and in any pattern desired. Top surfaces should be kept flush and the brick spaced for uniform joint thickness. The joints may be filled by sweeping sand over the surface, but in most cases a grout fill is preferable. The grout may be poured on the floor and swept into the joints until completely filled. If it is undesirable to have mortar or grout on the surface of the brick, the top should be given a brush coat of raw linseed oil or a clear parafin oil just before grouting and the grout poured into the joints and smoothed flush with the surface of the brick.

## 705. PATTERNS

Brick paving may be laid in any desired pattern with the brick laid either flat or on edge. A few of the more common patterns are shown on page 199, these being typical for either flat or edge setting, except that in the basket weave pattern each square will require only two brick flat as shown, or three brick on edge.

In R-B-M slabs requiring reinforcing bars at the top, the pattern is governed to a certain extent by the necessity for straight joints to receive the bars. Otherwise, there is no restriction on patterns that may be used.

## 706. CLEANING

Brick paving may be cleaned in the same manner as described in Chapter 3 for brick walls.

## 707. FINISHING

Where brick floors with grouted joints are to be used for finished surfaces, they should be carefully laid to a uniform surface and allowed to set hard. The surface may then be ground and waxed by the same method and equipment as is used on terrazzo floors.

## 708. BRICK ARCH FLOORS

The brick arch is an older type of brick floor construction and possesses great strength and resistance to fire. It consists of a 4″ brick arch built between steel floor beams spaced up to eight feet on centers. The bottom flanges of the steel are protected by flange blocks forming skews for the arch.

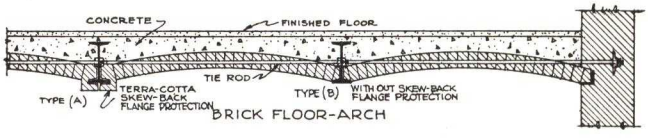



*Figure 24*

The arch is laid on wood centering, the ends of the brick butted together and the joints filled with grout. The arch is covered with a concrete fill leveled off for the floor finish.

Such a floor spanning 6 feet with a 4″ camber will carry a load of 300 to 400 pounds per square foot and the weight of the brick alone is about 40 pounds per square foot.

With the development of R-B-M, floors may now be constructed of reinforced brick masonry slabs instead of arches. Such slabs can be designed to carry the loads with equal fire resistance, considerable saving in weight and at lower cost. This type of construction is described in Chapter 10.

## 709. WALKS

The methods of laying brick walks are the same as described for floors except for a few precautions against exposure.

The brick should be hard-burned and low in absorption and it is advisable to follow the recommendations of the brick manufacturer in selecting brick suitable for this purpose.

Walks laid on a sand bed may become uneven, which with grass growing in the joints might be desired in some cases. To preserve an even surface, however, the brick should be laid on a ½″ cushion of cement and sand (1:4 mix) spread dry over a lean concrete base (1:8 mix) about 3″ deep.

The surface of all walks should have a slight crown or slope for drainage. If the walk is laid over clay or any water-retaining soil condition, provision should be made for the positive removal of water from under the paving.

To keep grass from growing in sand joints, a lean lime mixture may be used, consisting of one part dry hydrated lime to six parts dry sand. This dry mixture is swept into the joints. Salt should not be used as it may produce efflorescence on the brick. Gasoline will remove and prevent growths in the joints without discoloring the brick.

## 710. PORCHES AND TERRACES

The above directions apply also to the paving of porches and terraces. Attention is also called to Chapter 10 describing the several designs of R-B-M slabs suitable for such floors.

## 711. DRIVES

Brick paving for private drives is laid in the same general manner as for walks, with provisions for the heavier traffic.

The brick should preferably be set on edge because of their greater strength in that position.

Joints should be filled with grout to form a continuous slab and, if heavy truck traffic is expected, a vitrified paving brick should be used and preferably on a 3″ to 6″ concrete base. Expansion joints should be provided every 30 feet.

## 712. STEPS

Steps should be constructed on a firm soil base, forming horizontal beds and not sloped. They may be built up entirely of brick without concrete and preferable reinforced with light rods embedded in the mortar beds or grout filled joints. The firmness of the soil bed may be disregarded entirely, becoming merely a form, by supporting the steps entirely on side foundation walls built down below frost line, in which case the steps should be designed according to the principles of reinforced brick masonry design.

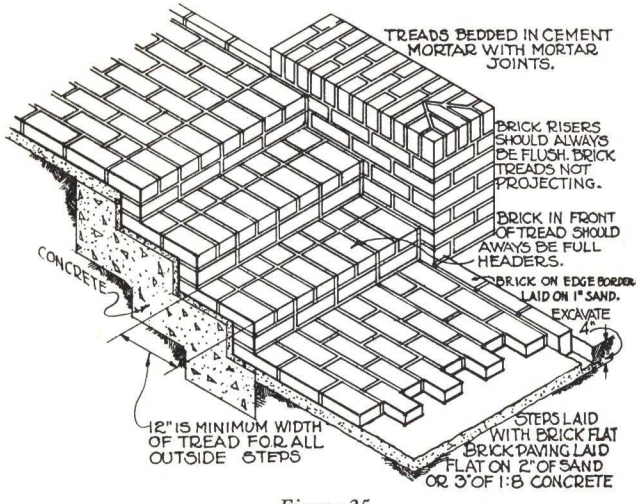



*Figure 25*

*Steps of R-B-M do not require concrete base.*

Treads of exterior steps should never be less than 12″ wide and should pitch forward with a slope of not more than ¼″ per foot for drainage. The front of treads should be formed of full length headers and not half bricks.

The joints of brick steps may be filled with cement mortar or cement-lime mortar but grout is preferable, particularly for the top surfaces to insure complete filling of the joints. Exposed joints should be tooled smooth or to a slightly concave surface. Brick surfaces may be kept free from grout as previously directed.

## 713. QUANTITIES

Assuming all joints to be ½″ thick, paving of brick laid flat require 4.5 brick per square foot, and on edge 6.2 per square foot. The quantity of mortar required may be obtained by reference to tables in appendix.

## 714. GARDEN WALLS

Brick harmonizes well with any garden, formal or otherwise, and in addition to beauty, brick masonry possesses the qualities of durability and permanence to be desired in outdoor structures.

Garden walls may be 4″ thick, if properly braced laterally. Pier and panel walls, described in Chapter 5, are economical to build and occupy but little space. The serpentine wall is also built 4″ thick, obtaining necessary lateral support from the curves in the plan.

Walls 8″ thick may be built solid or all-rolok and, stiffened laterally by 12″ x 12″ or 12″ x 16″ pilasters about 12 feet apart. Corners, offsets or curves in the plan serve the same purpose. All masonry structures should extend to a solid bearing below frost line and the necessity for a projecting footing depends more upon the nature of the soil than upon the load carried.

The tops of walls should be capped in a manner to prevent the entrance of water into the interior of the masonry. *This is an important detail in all structures* completely exposed to weather, as are such outdoor structures. Joints should be completely filled with mortar and tool or trowel pointed and flat surfaces should have a slight slope for drainage.

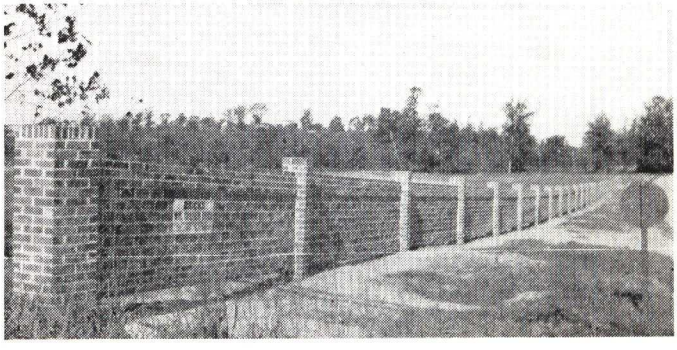


*Figure 26*

*An economical and attractive R-B-M garden wall.*

## 715. GARDEN STRUCTURES

The uses of brick for garden structures are unlimited and architectural design is unrestricted. Garden walls may be built solid or pierced, with light tracery. Garden seats and tables of brick are easily constructed of reinforced brick masonry. The main precautions to be observed are suitable foundations, resistance to penetration of moisture and, in the case of large areas, provision for expansion.

## 716. POOLS

Brick masonry presents the most suitable material for the construction of pools, both large and small. It is not affected by exposure to soil which may be acid or alkali, to frost action in the cold season, or to alternate wetting and drying. Brick may be used for the exposed surfaces as well as the structural floor and walls.

Regardless of size, reinforced brick masonry construction is recommended.

# CHAPTER 8
# MOISTURE CONTROL

## 801. MOISTURE PENETRATION

Difficulties due to moisture penetration through exposed brick masonry should be anticipated and avoided. The absorption of the brick has little to do with moisture penetration, the fault lying largely in the mortar joints and the prevention is entirely a matter of design and workmanship.

The following recommendations should be emphasized:

**(a) Design**

Cavity walls or hollow walls without through headers.
Adequate flashing (See Sec. 805).

**(b) Workmanship**

Full mortar joints.

**(c) Materials**

Low rate of absorption of brick when laid (wet highly absorbent brick).
Workable mortar of high water retentivity.

An investigation conducted at the National Bureau of Standards and reported in Building Materials and Structures, Report BMS7, included tests on 113 brick wall panels 4″, 8″ and 12″ thick built of three grades of brick, which were classed as nearly impervious, of medium absorption and of high absorption. The mortars used were mixed in the proportions of 1:0.25:3, 1:1:6 and 1:2:9 of cement, lime and sand, measured by volume. Two classes of workmanship were employed, "Ordinary" and "Inspected" as described in Section 315.

"The results of the tests indicate that it is not difficult to construct walls of brick masonry having a satisfactory resistance to the penetration of rain water . . . . The quality of the workmanship had a far greater effect on the performance of the walls than any other visible factor in the study. The importance of workmanship is illustrated by the fact that the walls of the same dimensions and of the same material showed the poorest or the best performance depending solely upon the quality of workmanship . . . . The best performance was obtained when the interior joints were well filled and the face joints were tooled . . . . Walls of workmanship A (Inspected), 4 inches in thickness and parged on the back with mortar, withstood the permeability tests about as well on the average as 8-inch walls of workmanship A. . . . Essential aids in obtaining walls resistant to moisture penetration were: The wetting of absorbent bricks before laying, the use of mortars of moderate or high water-retaining capacity and the use of bricks of high absorption as backing for bricks of low absorption."

For a detailed discussion of mortars, refer to Chapter 2.

## 802. STAINING

Staining is caused by the deposit of foreign materials such as iron rust, soot, etc., over the exterior surfaces resulting in yellow, brown, or red stains.

The possibility of removing such stains is questionable. The method of cleaning brick masonry is described in Section 325, but if this is not effective and other methods are tried, they should be applied with caution. Strong solutions may have a destructive action on the mortar and may cause further discoloration of the brick.

## 803. EFFLORESCENCE

The appearance of efflorescence on masonry walls is rightly a matter of concern. Aside from its unsightly appearance, efflorescence is a symptom of moisture penetration which, under certain conditions, promotes disintegration of the masonry. The discussion in this section is limited to clay products masonry.

Efflorescence on brick masonry walls is usually a light powder or crystallization, caused by water soluble salts, deposited on the surface upon evaporation of the water. Some of the salts frequently found in efflorescence are calcium sulfate (gypsum), magnesium sulfate (epsom salts), sodium chloride (table salt), sodium sulfate and potassium sulfate. There are two general conditions necessary to produce efflorescence: (1) soluble salts present in the wall materials, and (2) moisture to carry these salts to the surface.

Soluble salts may be present in brick, hollow tile, concrete units, mortar or plaster. Only a very small percentage of the brick and hollow clay tile produced will contribute to efflorescence. In fact, tests at the Bureau of Standards on several hundred samples of brick, ranging from under-burned to well-burned, indicated that 83%, or 5/6 of all the brick samples would not be the source of efflorescence. If the results were weighted according to production, the percentage free from efflorescence would be greater. It is probable that more than 90% of the production of structural clay products in this country will not tend to effloresce. Second-hand brick, because of its uncertain origin and previous contact with mortar and plaster, may frequently be a source of efflorescence. Concrete blocks usually contain soluble salts and are apt therefore, to be the cause of efflorescence. Mortars and plaster are frequently the source of efflorescence, due to one or more of the ingredients. Portland cements contain soluble salts, and so do some limes and certain sands.

The wick test for efflorescence in building brick is described in Research Paper 1015 by Messrs. McBurney and Parsons of the National Bureau of Standards, a resumé of which appears in Chapter 2.

Another test for efflorescence in a brick may be made by chemical analysis of the brick. This test, however, is not made as readily as the wick test. L. A. Palmer states in Technologic Paper No. 370 of the Bureau of Standards, that a brick containing 0.05 per cent or less of water-soluble sulphuric anhydride in its outer $\frac{1}{8}''$ of exposed surface, is not apt to contribute to efflorescence of a wall.

To determine the source of efflorescence, all materials should be tested.

When efflorescence makes its first appearance on a wall, it may be possible to determine the source. If it appears at the edges and not near the center of the brick, it is probably the mortar which contains the soluble salts and not the brick. If the efflorescence covers the whole brick, it is likely that both the brick and mortar are the cause of it; while efflorescence near the center of the brick and not near the edges indicates that the brick is the probable source. A uniform coating frequently appears on a new building as the result of excess water during construction. After the walls have had an opportunity to dry thoroughly, a final cleaning, or sometimes a few rains, will wash away the efflorescence, and it seldom appears again.

Since moisture is necessary to carry the soluble salts to the surface of the masonry, efflorescence is evidence that moisture has entered the wall and there is faulty construction that should be corrected. An investigation should be made of the structure to determine how and where the moisture enters.

Wet walls may be due to one or more of the following: defective flashings, gutters and downspouts, faulty copings or improperly filled mortar joints. The location of efflorescence does not necessarily mean that water is entering the wall at that point, but frequently provides a clue to the source of the trouble. For example, streaking from the top of a wall down or, at times, patches some distance from the top, would point to defective copings or gutters. Efflorescence under windows would put the sills and the

caulking around the frame under suspicion. Efflorescence on the courses of a foundation, close to the ground, especially with a rather porous brick, would indicate ground water drawn up by capillary suction. A single patch of efflorescence on a wall, with no observable relation to masonry openings, may sometimes be explained on the grounds of a badly defective mortar joint or by a projecting brick forming a water table. In any case, the general principle is that this spot of efflorescence represents a portion of the wall unduly wetted.

Faulty flashings, gutters or downspouts should be repaired. Copings should be laid with thin, but well-filled mortar joints, with weathered or rodded tooling. Non-corrosive metal or bituminous flashing should be placed directly under copings, cornices, chimney caps, sills and projecting courses of masonry. Improperly filled joints in exposed walls should be raked out and repointed with a plastic mortar of approximately the same mix as used in the original work, but allowing the mortar to partially set for 30 to 45 minutes before using. Avoid the use of cement, lime or sand that may tend to cause efflorescence. Proper caulking around window and door frames, raking and repointing mortar joints in sills may be necessary.

Dampness in foundation and lower part of walls above grade may be caused by lack of, or faulty: (a) drains along footings, (b) waterproofing on outside of foundation walls, and (c) dampproofing in masonry course immediately above grade. The first two, (a) and (b), may be corrected after construction, but a dampproof membrane is practically impossible to insert after a wall is built. If conditions warrant it, a remedy may be obtained by drilling ½″ holes on 4 or 5 ft. centers in a horizontal mortar joint a few courses above grade and to a depth of approximately 4″ or sufficient to reach the first vertical joint back of the face. Grout containing 2% of ammonium or calcium stearate by weight of cement and lime is then forced under pressure into alternate holes until the grout appears at the intermediate holes. Holes are sealed with regular mortar.

## 804. REMOVING EFFLORESCENCE

Water applied with stiff scrubbing brushes will frequently remove efflorescence. If this does not do a complete job, apply water first, then scrub with water containing not more than 10% of muriatic (hydrochloric) acid and, immediately thereafter, rinse thoroughly with plain water. It is sometimes desirable to give surface a final washing with water containing approximately 5% of household ammonia. For further discussion of cleaning clay products masonry, see our Technical Notes Bulletin No. 9.

To sum up, efflorescence in masonry construction can be prevented by the use of suitable materials, proper design and good workmanship. The choice of materials has been discussed somewhat, but more emphasis should be placed on design and workmanship. It should be remembered that a correct design and good workmanship will eliminate efflorescence, frequently with materials that might otherwise effloresce in abundance.

## 805. FLASHING STRUCTURAL CLAY MASONRY

### (a) PURPOSE

At the outset, it might be stated that no flashing at all is better than poor flashing. Improper materials may be destroyed by corrosion or damaged by movement of the structure, and even good materials may be improperly installed. The result is that instead of excluding or controlling moisture, defective flashings tend to collect it in a manner to cause serious damage to the structure.

Unless the complete protection of the structure against moisture penetration justifies an expenditure sufficient to guarantee adequate and effective flashing, it is good economy to omit the flashing altogether and to rely on maintenance (frequent repointing of mortar joints in horizontal surfaces and other vulnerable spots) to prevent the

entrance of water into the wall. To merely specify flashing and then trust to chance that it will be effective has proved most disappointing in many installations.

External or applied flashings are those which prevent the absorption by vertical masonry construction of water accumulating on relatively flat intersecting surfaces. Such flashings usually consist of two members, the base flashing which forms a part of the covering of a flat surface and turns up against an intersecting vertical surface, and the cap or counter flashing which is built into the vertical surface and turned down over the base flashing. Internal or through flashings are those built into and usually concealed within the masonry to control the travel of moisture and direct it to the exterior surface. It is desirable that some external flashings become also through flashings, particularly in the case of counter flashings. In general, the ends of through flashings should be turned up to prevent drainage into the wall, and the back should be turned up if below the roof line and turned down if above the roof.

### (b) WHERE FLASHING IS REQUIRED

The vulnerable points of a masonry structure where flashing is necessary or recommended for good construction will be described in the order of their occurrence in the progress of construction. They are at the grade line, the sills and heads of openings, spandrel beams, the intersection of vertical and horizontal surfaces, and at parapet walls.

### (c) FLASHING MATERIALS

Flashings are usually formed of sheet metal or bituminous membrane materials, the selection being greatly influenced by cost. The cost of installation is practically the same for any material, so the cost variation lies mainly in the material itself. The cost of replacing defective flashings will probably greatly exceed the original saving on material, therefore it is advisable to select a permanent material for the original installation.

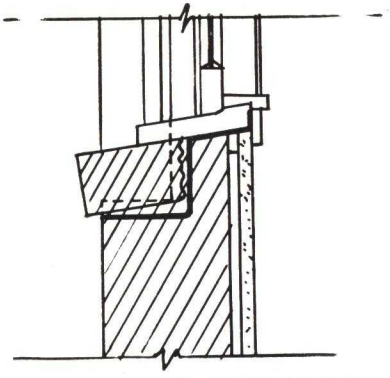
Tin and galvanized iron may be used for flashings, but they are subject to corrosion and require maintenance. Other metals suitable for flashings are copper, lead, zinc and aluminum. Of these, copper is generally preferred for good work as it does not deteriorate when exposed to weather or to masonry materials. The principal objection to copper is that, where exposed to weather, it may stain or discolor light colored masonry surfaces. This may be prevented, however, by the use of lead coated copper at such places. While lead is attacked by cement mortar and fresh lime mortar, this action is negligible near the masonry surface because the mortar carbonates quickly and the lead coating remains effective. Copper sheets are available for through flashing, formed with corrugations and indentations to provide mechanical bond with the masonry and interlocking joints where the sheets lap.

Bituminous membrane materials are composed of asphalt or tar saturated fabrics and they are usually laid in and covered with a bituminous waterproofing compound applied with a trowel or brush. If these materials are not permanently insoluble in water they also may produce stains or discoloration on the masonry and should not be used where this would be objectionable.

### (d) DAMP CHECK

Moisture from the ground may be absorbed by exterior foundations and travel upward by capillary action into the wall above. This should be prevented by the installation of a damp check consisting of a continuous through flashing covering the full thickness of the wall and having tight end joints. It should be placed about 6″ above the finished grade, platforms, or other exposed surfaces as further protection against moisture from rain splash or snow. This flashing, if of metal, may be increased in width to project over one or both faces of the wall to form a termite shield. Slate has been fre-

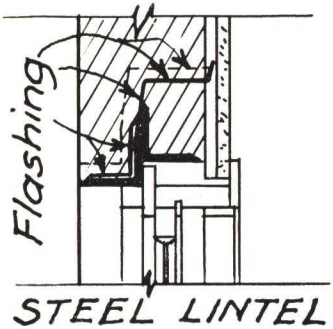
quently used for this purpose and is satisfactory as a moisture barrier if the edges are well lapped. It must be bedded in mortar and, with the lapped joints, increases the thickness of the mortar bed to an extent that it may mar the appearance of the wall. In case of uneven settlement it may also be broken, in which case dampness may penetrate the upper wall.



SILL FLASHING

(e) **SILLS**

Unless made of an impervious material in one piece, all sills should be provided with through flashing. This flashing should be formed to fit under and behind both the masonry and the wood sills. The ends should extend beyond the jamb lines and be turned up at least 1″ into the wall to keep any accumulated moisture from entering the wall at these points. If the sills are of cast stone, the ends should be full height, forming closed end pans.




STEEL LINTEL



R-B-M LINTEL

(f) **HEADS**

The heads or lintels of openings should be treated in a similar manner with through flashing. For steel lintels, the flashing is placed under and behind the facing wythe of masonry and over the top of the lintel. Bend the front (exterior) edge of the flashing down over the edge of the angle to form a drip and thus eliminate moisture entering and causing the angle to rust. A bituminous coating on top of the outer edge of the angle may help seal this space. For reinforced masonry lintels that are not grout-filled, the flashing is placed between the masonry and the frame, the front edge turned down as a

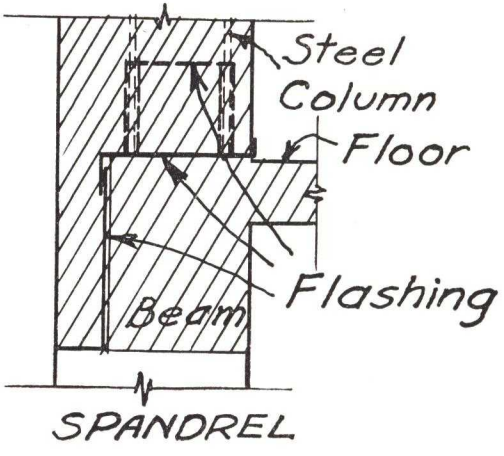
drip over the face of the frame. In all cases the ends should be formed as for sills and the back turned up one inch against the inner face of the wall or of the furring. (RBM and other masonry that is grout-filled requires only a damp-check).

In cavity walls, the same methods are followed, the flashing sheet extending across the cavity and into the inner wythe one or two courses higher than at the outer wythe.

### (g) SPANDRELS

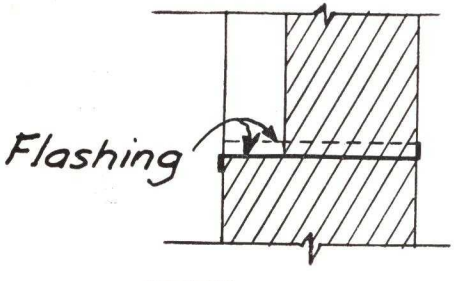
In skeleton framed structures, the spandrels should be flashed continuously at the beams. The flashing should start under the facing wythe at its supporting member, extend upward behind the facing wythe and over the top of the beam, which is usually the rough floor level, and turn up at the inside face of the masonry wall or furring. This portion may be formed in two or more overlapping sections to simplify installation. It should also turn up about 6″ high around columns above the beam, allowing ¼″ clearance between column and flashing metal, with tight bottom joints, and the top sealed with mastic.

Copper is the most satisfactory material for this purpose, but it is necessary that the sheets be carefully fitted, especially at columns, and all joints soldered tight.

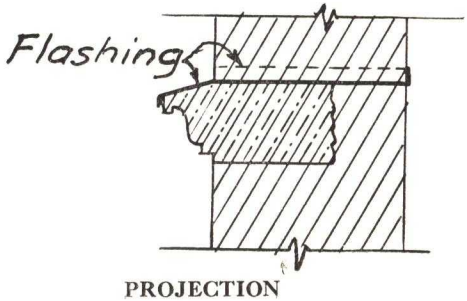



### (h) PROJECTIONS

All projections and recesses from exposed face of a wall tend to retain rain water and snow and should, therefore, be designed with a slope or wash on top to drain off the water and, if possible, a drip to throw the water clear of the wall surface below. They should also be flashed, preferably over the top, with the outer edge of metal bent down




RECESS




PROJECTION

and folded as a drip and the back extending through the wall and turned up against the inner face of the wall if below the roof flashing. If made of cast stone, all courses, either projecting or flush, should have through flashing in the mortar bed below the cast stone, or this flashing may be formed as closed end pans as described for cast stone sills.

### (i) ROOFS

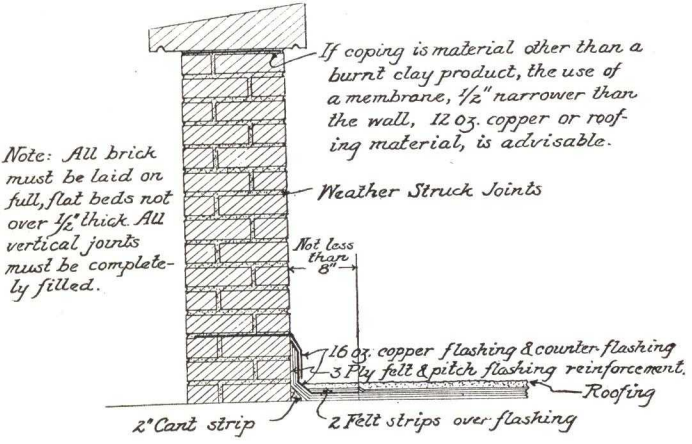
The design of base flashing depends upon the kind of roofing material used. Where the base flashing is of metal, the counter flashing should also be of metal extending through the wall and turned down overlapping the base flashing. At chimneys, it should extend through the chimney walls and turn up at least 1″ against the flue lining.

The most satisfactory method of flashing built-up roofing is by extending it into raggle blocks built into the wall, and through flashing should be placed in the mortar bed on top of the raggle block. Without raggle blocks, the roofing material may form the base flashing, in which case the through flashing, extended and bent down, also becomes the counter flashing.

### (j) PARAPETS

Parapet walls should be built of good materials and faced with the same quality material as on the exposed side. This applies as well to other roof structures so frequently neglected because they are not exposed to view. The backs of parapets should not be painted or coated in any manner, but instead left free to dry rapidly. If they must be covered, the material used should be erected free from the wall in a manner to permit circulation of air between the covering and the masonry.

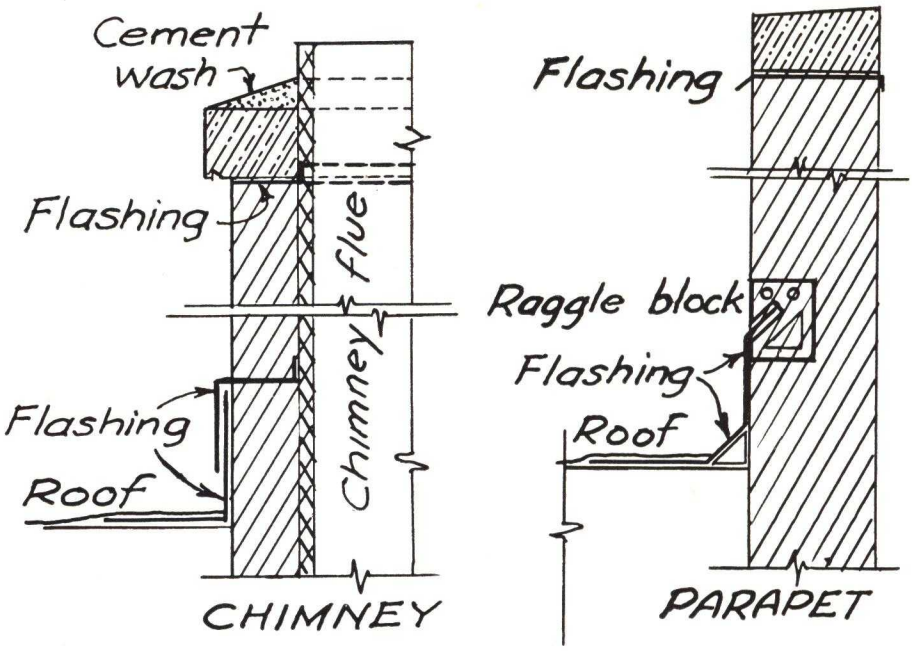
In all cases, through flashing should be placed in the mortar bed under the coping unless it is of an impervious material with water tight joints. Where the coping provides suitable drip the flashing may stop at the wall surface, otherwise it should be turned down ¼″ against the wall on both sides. The same treatment should be applied in topping out or finishing similar exposed structures, such as chimney tops and piers.




### (k) CHIMNEY TOPS

Through flashing, as described for parapets, should be placed under chimney tops, cut out for flues and turned up tight against the flue linings.

Undoubtedly, the most important feature in masonry construction to prevent moisture penetration is the *complete filling of all mortar joints.*




## 806. DAMPNESS ON INSIDE WALLS

### (a) CAUSES

Technical Bulletin TIBM-25 of the National Bureau of Standards contains the following regarding the value of plastering.

"The occupant of a building is constantly aware of the quality of plaster on walls and ceilings through daily impressions, made subconsciously upon his mind perhaps, by which he becomes cognizant of whatever faults or merits it may possess. To him, its obvious value is that of decoration, although it has other functions which though none the less important, are frequently lost sight of."

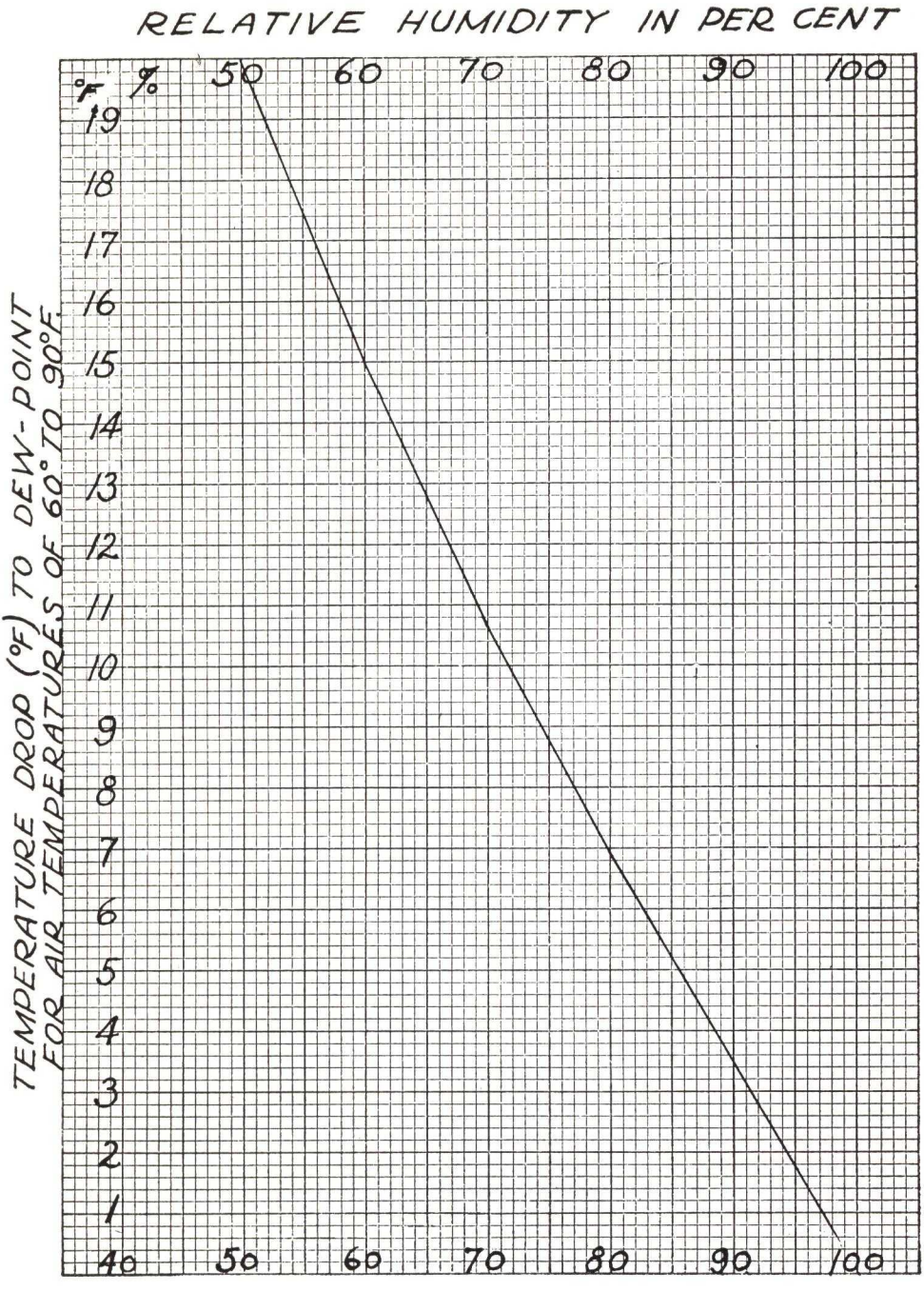
Moisture on interior walls even in small quantities tends to destroy wall decorations and in larger amounts may result in the failure of the plaster. Regardless of the actual damage occasioned by the moisture, its appearance inevitably results in a dissatisfied tenant and frequently the elimination of the cause of the trouble is an inconvenient and expensive operation.

Damp inside walls are not peculiar to any type of construction nor to any building material but usually result from one of the following causes: leakage, through exterior walls or roof; condensation; or presence of hygroscopic salts in the plaster.

When the moisture appearing on the inside of a wall is due to leakage, the water has percolated through the plaster. This action softens the plaster and tends to loosen it from the lath or surface to which it is applied.

### (b) CONDENSATION

Atmospheric air is a mixture of dry air and water vapor. At a given temperature air is saturated when the space occupied by the mixture holds the maximum possible weight of water vapor at that temperature. The amount of water vapor necessary to saturate air at constant pressure depends upon the temperature, the higher the tempera-


RELATIVE HUMIDITY IN PER CENT
°F
%
50
60
70
80
90
100
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
40
50
60
70
80
90
100
TEMPERATURE DROP (°F) TO DEW-POINT
FOR AIR TEMPERATURES OF 60° TO 90°F.

ture the more water vapor will be required. If saturated air at a temperature of 50° for instance is warmed to a temperature of 70° the mixture is no longer saturated but will absorb additional water vapor. However, if an unsaturated air is cooled at constant pressure, a temperature will be reached at which the air is saturated. This temperature is called the dew point and if the mixture is cooled below the dew point, water will condense from the air. Dew which occurs in the early mornings during the warmer months in many localities is one of the commonest examples of the effect of cooling unsaturated air to a temperature below the dew point or to a point where the water vapor which the air contains begins to condense.

The water vapor in air is called Humidity and Relative Humidity is the ratio of the amount of water vapor which a mixture contains to the amount required for saturation at a given temperature. Obviously for a fixed amount of water vapor the Relative Humidity will vary with the temperature, increasing as the temperature is lowered and decreasing as the temperature rises.

While the dew point depends upon the amount of water vapor in the air and is the temperature at which the water vapor present is sufficient for saturation, there is also a practically constant relation between dew point and Relative Humidity for a considerable range of temperatures. That is for a Relative Humidity of 50% the difference between air temperature and dew point is approximately 20° for any air temperature from 60° to 90°. Similar relations hold for other Relative Humidities. A discussion of the reason for this relationship may be found in the 1939 edition of the American Society of Heating and Ventilating Engineers Guide, Page 9.

On the curve of the accompanying chart, the difference in temperature between the air and the dew point (temperature drop) is plotted for Relative Humidities from 40% to 100%. It will be noted from this curve that for Relative Humidities above 80% a drop in temperature of 6.8° or less is sufficient to cause condensation. High Relative Humidities, however, usually occur during the summer months when the difference in temperature between the air on opposite sides of a wall is usually small, probably 10° or less. For walls below grade the temperature difference of the two sides of the wall may amount to 20° or more.

If condensation occurs on walls it will appear during periods of high Relative Humidity or during extremely cold weather when there is a large difference in temperature between inside and outside air.

The ventilation standards of the American Society of Heating and Ventilating Engineers provide that for "occupied spaces — the Relative Humidity shall be not less than 30% nor more than 60%".

For a Relative Humidity of 60% the temperature drop necessary to produce condensation is 15°, however, in residences the Relative Humidity may reach a value above 60% due to the introduction of water vapor into the air in the form of steam from cooking or other sources. If condensation occurs it may be eliminated by:

(1) Reducing the Humidity of the air. This may be accomplished by adequate ventilation if the high Humidity is caused by conditions inside the building.

(2) Increasing the temperature of the surface upon which the condensation occurs. Probably the simplest means of increasing the temperature is to increase the movement of air over the surface.

(3) Increasing the heat resistance of the wall. This is usually done by the addition of an air space back of the plaster.

The following table gives the differences in temperature between the inside surface of various types of walls and the inside air for temperature differences between inside and outside air ranging from 10° to 70° F. These figures are based upon the conductivi-

ties and conductances of materials listed in the 1939 edition of the American Society of Heating and Ventilating Engineers Guide.

### TABLE NO. 25

**Difference in Temperature between Inside Air and Inside Wall Surface for Various Total Differences in Temperature between Inside Air and Outside Air**

| Difference in temperature between inside and outside air — °F. | 10 | 20 | 30 | 40 | 50 | 60 | 70 |
|---|---|---|---|---|---|---|---|
| | Difference in temperature between inside air and wall surface — °F. | | | | | | |
| 8″ Solid brick plastered | 3 | 5½ | 8½ | 11 | 14 | 16½ | 19½ |
| 8″ Solid brick furred and plastered | 2 | 3½ | 5½ | 7½ | 9 | 11 | 12½ |
| 4″ Brick 4″ tile plastered | 2½ | 5 | 7½ | 10 | 12½ | 15 | 17½ |
| 4″ Brick 4″ tile furred and plastered | 2 | 3½ | 5 | 7 | 8½ | 10 | 12 |
| 8″ Hollow tile plastered | 2½ | 4½ | 7 | 9½ | 12 | 14 | 16½ |
| 8″ Hollow tile furred and plastered | 1½ | 3½ | 5 | 6½ | 8 | 10 | 11½ |

Temperature differences are less for 12″ walls.

From the above table it will be noted that for a difference in temperature between inside and outside air of 70° the inside temperature of an 8″ solid brick wall plastered direct would be 19½° below the temperature of the inside air. Referring to the curve, a temperature drop 19½° will not cause condensation for Relative Humidities of 50% or under.

Temperature ranges in various localities may be obtained from the National Weather Bureau and if the designer can determine the Relative Humidity that will be maintained within a building the above table may be used in conjunction with the "Temperature drop" curve to select a type of wall construction which will be free from condensation.

### (c) HYGROSCOPIC SALTS IN THE PLASTER

Hygroscopic salt is a substance which will absorb water from the air. One of the best known of such salts is calcium chloride which is frequently used to keep down the dust on roads, parking lots, and similar areas and is sometimes used in curing concrete. If an appreciable quantity of ordinary table salt (Sodium Chloride) is introduced into plaster, the calcium either as lime or gypsum will combine with the salt and form calcium chloride. In a rather dilute form it is not as active as the pure calcium chloride, however, during periods of high Relative Humidity, it will absorb sufficient moisture from the air to stain the wall paper or other decoration which may have been applied to the plaster.

If the moisture which appears on a wall is due either to condensation or to hygroscopic salts in the plaster, the moisture comes from the air inside of the room and does not penetrate the plaster. Consequently, the danger of falling plaster is minimized and the condition of the plaster may serve as a guide to the source of the moisture.

### 807. WATERPROOFING

In damp or wet soils, it is advisable to waterproof basement walls. This is less expensive during construction than after a fill has been made.

An inexpensive method of waterproofing basement walls consists of plastering the exterior of walls with cement mortar, either with or without an addition of calcium stearate (in an amount equal to 3 per cent of the weight of the cement) or its equivalent in ammonium stearate. Other forms of insoluble soaps may be substituted for the stearates. The chief defect of this form of waterproofing is its rigidity. Any structural defect causing a crack in the wall may crack the plastering. It may be adequate, however, if the wall is subjected only to occasional dampness and not to water under pressure.

Another method consists in the application of a coating of bituminous mastic, being a mixture of asphalt or coal tar pitch, a solvent and an inert filler such as silica sand or asbestos.

Only the membrane types, however, can be considered absolutely watertight. Three or more layers of cotton fabric or felt bonded together with asphalt or coal tar pitch applied hot will resist water under considerable pressure. Furthermore, such a membrane is capable of yielding without breaking.

Waterproof wall coatings should be continuous from the top of the footing to the finished grade line. As a further precaution, a damp check may be placed at the top of the footing, covering the full width of the wall and connected to the exterior waterproofing. Basement floors may be waterproofed by placing the membrane coating on the concrete base slab and laying the brick in a mortar bed over the coating.

## 808. TRANSPARENT "WATERPROOFERS"

No surface waterproofing is capable of filling holes in masonry or correcting structural defects. Surfaces requiring treatment should, therefore, be put in good repair. The majority of transparent waterproofers are:

Paraffin or very heavy mineral oils in solution in light mineral spirits; metallic soaps (aluminum, zinc, etc., salts of fatty acids); varnishes, usually mixtures of organic oils and gums; materials in water solution, intended to penetrate the pores of brick, stone and mortar and form seals, as for example, water glass (sodium silicate), fluro-scilicates, etc.

The use of transparent "waterproofing" materials is probably justified only when defects in construction have been made good and when it is then evident that the only moisture entering a wall is entering through the vertical face and is due to the porosity of the bricks or mortar or both, and not to defective joints (no transparent "waterproofer" will stop a hole) and this moisture is sufficient to cause and continue to cause efflorescence.

The choice of a suitable "waterproofer" can probably best be made on the basis of selecting that brand or type which actual experience in a given locality has shown to be the most permanently effective in withstanding local climatic condition. Current opinion concerning this question is varied and often conflicting. Laboratory investigation does not thus far, justify positive conclusions.

# CHAPTER 9
# REINFORCED BRICK MASONRY
## General Discussion

### 901. DESCRIPTION

As a potential type of construction, Reinforced Brick Masonry is receiving increasing recognition and offers many new and practical uses for that proven building material of the ages. Except in the unusual case, brickwork has generally been considered as suitable only for carrying direct compressive stresses, but in Reinforced Brick Masonry, due to a scientific combination or reinforcing steel with plain brickwork, there results a type of construction capable of carrying heavy loads in flexure and in shear, as well as in direct compression, thus meeting the requirements of modern structural engineering practice.

Reinforced brick masonry is not a patented form of construction, nor does its design involve the use of any new and untried materials. Standard commercial brick of good quality, are suitable when laid in the proper kind of mortar, and special shapes are unnecessary, although in some instances they may be found desirable. New shapes are being produced as fast as demand arises for their manufacture.

The reinforcing steel employed is of the same kind and quality commonly used for reinforced concrete and thus it may be seen that the constituent materials required for Reinforced Brick Masonry are those now in common use and everywhere readily available.

The introduction of reinforcing steel into brickwork offers no obstacle to the builder nor is the daily output of the mason materially reduced. The reinforcing bars are introduced into the masonry by placing them either in the horizontal or in the vertical mortar joints or in both. Bent up bars and vertical stirrups are easily introduced and effectively resist web stresses. Occasionally a bed of mortar, of suitable thickness, is laid first and the horizontal reinforcing steel is placed therein. For slab construction, the use of expanded metal, electrically welded fabric or other similar light metal shapes may offer some economy and prove advantageous.

Since Reinforced Brick Masonry may be used in practically every type of engineering structure, it will be impossible to detail all of its applications in this volume. It is believed, however, that the general text, supplemented by design formulas, tables and charts will be found sufficiently complete to enable the designer and builder to proceed with assurance and safety. Scientific research has added materially to our knowledge of this subject and will continue to assist in the establishment of standardized practice.

The principles of Reinforced Brick Masonry design are the same as those commonly accepted for reinforced concrete design and identical formulas may be used. Therefore, the experienced Architect or Structural Engineer will have no difficulty in extending his knowledge to the requirements of Reinforced Brick Masonry design.

### 902. HISTORICAL BRIEF

Reinforced brick masonry is not of recent origin. It was discovered in principle, over 100 years ago, by that eminent British engineer, Marc Isambard Brunel, once chief engineer of the City of New York and later knighted by Queen Victoria in recognition of his genius in planning and constructing the Thames River Tunnel.

Although Sir Marc first proposed the use of reinforced brick masonry as early as 1813, as a means of strengthening a chimney then under construction, it was in connection with the building of the Thames Tunnel, in 1825, that he made his first major

application of its principles. As a part of the construction program for that monumental work of engineering, two brick shafts were built, each 30 inches thick, 50 feet in diameter and 42 feet deep. Mr. Richard Beamish, F.R.S., a resident engineer at the tunnel, describes this shaft as having been reinforced with 48 wrought iron bolts, 1 inch in diameter, built into the brickwork and attached to wooden curbs at the bottom and top with nuts and screws, "thus binding the work into a solid mass." Iron hoops, nine inches wide and one-half inch in thickness, were also laid at intervals into the brickwork as the building of the shaft progressed. When the shaft had attained a height of 42 feet, it was caused to sink by excavating the earth from its interior, using what is now commonly known as the open method of caisson construction. The strength and cohesion of the brickwork in the shaft was severely tried, due to unequal settlements from time to time, caused by the presence of boulders and by the varying characteristics of the soil encountered. It stands as a matter of record however, that the method of reinforcing the shaft was so successful that no injury to the shaft resulted.

When the shaft for the opposite shore was built, it was reinforced in the manner outlined above for its entire height of 70 feet. In the memoirs previously noted, Mr. Beamish states: "The fortunate combination of iron ties with bricks and cement in the construction of the shafts of the Thames Tunnel suggested to Sir Marc the idea of establishing a general principle of construction which should unite to an eminent degree, utility with economy."

Accordingly in 1836, Sir Marc built what is known as the "Nine Elms beam." It was an inverted "T" section, 57 inches in total depth. The lower part of the beam was 24 inches wide and 18 inches high, and the upper part or stem was 39 inches high and 19 inches wide. Its clear span was 21 feet 4 inches. The beam was reinforced by 17 pieces of hoop iron 1/16 of an inch thick by 1¼ inches wide. On this beam a centrally located superimposed load of 23,829 pounds was placed and the beam stood thus loaded for a period of two years. It was finally tested to destruction in 1838 and failed under a total live and dead load of 68,326 pounds. The cause of failure was excessive tensile stress in the hoop iron reinforcement, most of the reinforcing being broken by the test. Roman cement, then just coming into use, was employed in the construction of this beam. The results of this experiment were reported to the Society of Engineers (England), and the discussion which resulted is most interesting. It was claimed by many members of the Society that the superior strength developed by the beam was due entirely to the use of Roman cement in the mortar. In fact, it was this skeptical attitude of the general engineering profession that kept reinforced brick masonry from making the rapid strides it so rightfully deserved. Further experiments were conducted by Brunel and others, some of which are hereinafter mentioned, but it was mainly individual effort and at personal expense, with little or no encouragement from the scientific fraternity. Engineers and scientists would not become reconciled to the theory that two materials, such as masonry and steel with widely different physical properties, could act together as a structural unit. Of course, engineers today are familiar with this phenomenon due to the wide spread use of reinforced concrete.

At about the same time (1836) Sir Marc erected at the Thames River Tunnel Yard, an experimental reinforced brick masonry structure consisting of two cantilevered half arches. His principal thought was to build an arch without the use of centering and it is of interest to note that no falsework or centering was used and that all brick laying was done from the completed portion of the structure as the work progressed.

One semi-arch of the structure projected out from the supporting pier a distance of 60 feet and the other a distance of 30 feet. On the 30-foot semi-arch a counterweight of 62,700 pounds was placed to balance the weight of the longer arch. The supporting pier was 10 feet by 4 feet in cross section. The arches were built entirely of brickwork and were cantilevered 15 feet longitudinally on each side of the pier with plain brick-

work. From these points on both arches, the work was of reinforced brick masonry, built by placing hoop iron in the top brick courses, thus providing the necessary reinforcing steel to resist cantilever action.

After standing for two years, the structure collapsed during a storm, due to undermining of the supporting pier foundation by the construction of adjacent work.

In 1837, Colonel Pasley of the Corp of Royal Engineers, conducted a series of tests on reinforced brick masonry beams and reported results comparable to those obtained by Brunel. Pasley's idea was to settle definitely the then current argument as to whether or not the flat hoop iron really strengthened brick beams. He built three beams in all, each 18 inches wide, 12 inches deep and of 10 foot span. One beam was built without reinforcement with the bricks laid in pure cement; the second beam also was laid in pure cement, but the beam was reinforced with five pieces of hoop iron. In the third beam, reinforced in the same manner as the second beam, the brick were laid in a mortar composed of one part of lime and three parts of sand. The first beam failed at a load of 498 pounds; the second beam carried 4,723 pounds, and the third beam failed at between 400 and 500 pounds, and thus was definitely settled the dispute which had been current for some time.

These beams were four brick courses in depth. Two of the five pieces of hoop iron were placed in the top mortar joint and were therefore located above the neutral axis. In the middle joint, a single piece of hoop iron was placed and, due to its position slightly below the neutral axis, was of little help in resisting tension. The remaining two pieces of hoop iron were placed in the lower mortar joint and carried the tensile stress. An examination of the second beam, after failure, showed that these two latter pieces of hoop iron were broken, thus proving that the metal had carried the full tensile stress and that the unit stress had exceeded the ultimate strength of the hoop iron. It may be observed from this test that the addition of two hoop iron strips (approximately 1/16 inch by 1¼ inch) increased the strength of the plain brick beam 4225 pounds or 848%. It may also be observed that the addition of reinforcement in the third beam (built of a mortar composed of one part lime to three parts sand) added nothing to its strength, a result verified within recent years which points definitely to the conclusion that a lime-sand mortar must never be used for reinforced brick masonry.

In 1851, at the Great Exposition in London, a test beam was built, identical in span and comparable in cross section and reinforcing to the Nine Elms beam, using hollow bricks with one longitudinal hole running the full length of the brick. This beam carried 12,514 pounds more live load than the Nine Elms beam, but it should be noted that in this beam the bricks were laid in Portland cement mortar, whereas in the Nine Elms beam, built and tested by Brunel, Roman cement mortar was used.

English technical publications of that period mention the use of reinforced brick masonry in connection with the construction of several large buildings and Mr. Isambard Kingdom Brunel, son of Sir Marc Brunel, built one of his most beautiful brick arches with hoop iron set into the brickwork in the same manner his father's experiments had proved so successful.

In 1872, Mr. N. B. Corson, in his discussion of "The Strength of Brickwork", called attention to the fact that little progress had been made during the 14 years between 1837 and 1851 in studying the strength of brick beams until the London Exposition beam was tested in 1851. From an analysis of tests made, Mr. Corson deduced a value for the tensile strength of brick masonry as shown by previous tests and developed a formula for the design of lintels. This appears to be the first recorded technical discussion of stresses in reinforced brick masonry beams and of related mortar strengths.

During the period from 1852 to 1905, frequent practical use was made of reinforced brick masonry in actual construction in several European countries and to a lesser degree

in the United States. As had previously been the case, practically all of the reinforced brick masonry construction was done independently by individuals scattered all over the world. There was little, if any, exchange of ideas or coordination of information. Each of these pioneers in R-B-M had faith in its possibilities and developed it according to his own particular tests or requirements. The more popular uses of reinforced brickwork were in high thin walls, columns, flat brick arches, silos and storage bins, large chimneys or stacks, water tanks and reservoirs and in thin wall sections as facing and forms for mass concrete in dams and retaining walls. These structures have given exceptional service and have substantiated the faith of the designers in this type of construction.

The sponsors of reinforced concrete, in its early days, referred with pride to reinforced brick masonry as practical examples of the use and possibilities of the combination of steel and masonry in construction. In the 1904 edition of a textbook on "Reinforced Concrete", written by Charles Marsh, a reference and details may be found of the use of reinforced brick masonry in the Church of St. Jean de Montmarte, Paris, designed by M. A. de Boudot. The exterior walls are only 4½ inches thick, reinforced with wires vertically through holes in the bricks and horizontally in the mortar joints. One portion of one side wall has an unsupported length of 29 feet 6 inches for the full height of the wall (115 feet). On the other side of the church there is a wall 38 feet 4½ inches high with an unsupported length of 65 feet. The upper floor of this structure is supported by reinforced brick masonry columns 17⅜ inches square, spaced 39 feet 6 inches on centers.

Major E. S. Roberts, R. E. (1904) describes the application of reinforced brick masonry to cylindrical water tanks. This paper is an interesting contribution to the literature on the subject. Actual construction details and design data are given.

In "Clayworker", January, 1931, Professor R. F. Tucker, Massachusetts Institute of Technology, reported that he had in 1905, made use of a 4-inch reinforced brick masonry facing as forms within which to cast the concrete for a dam built near Ithaca, New York. The use of reinforced brick masonry in the manner described is said to have effected a material saving in cost.

In 1912, the Expanded Metal Company of England made tests on R-B-M slabs, walls, beams, and on circular construction, in which expanded metal was used as reinforcement.

In 1913, Hugo Filippi built and tested one beam, 8 inches wide by 14 inches deep (effective depth, 11¼ inches) and of 12'-0" span length. The reinforcement consisted of five (5) ½ inch square corrugated bars. No stirrups or bent bars were used. The mortar consisted of one part of Portland Cement, three parts screened Mississippi River torpedo sand and approximately 20% of slaked lump lime. Red, side cut, clay bricks, purchased in Memphis, Tennessee, were used. The beam was loaded at the quarter points and failure was by diagonal tension at a total applied live load of 18,400 pounds. The deflection, immediately before failure was ⅞ of an inch. The age of the beam was 41 days. Because of intermittent freezing weather during construction, the bricks were not wetted before laying.

In 1919, Mr. L. J. Mensch, C. E., of Chicago, tested four beams on a span length of 7 feet 6 inches. The beams were three brick courses deep and the reinforcement in two of the beams consisted of four ¼ inch straight rods laid in a ¾ inch bed of mortar, below the lowest course of brick. In the remaining two beams, six ¼ inch rods were used — three straight bars in a similar ¾ inch bed of mortar and three bent bars in a longitudinal vertical mortar joint between the bricks in the lowest brick course.

The ultimate concentrated live load at failure, applied at the center of the span, ranged from 4170 pounds to 5225 pounds. Two beams failed by diagonal tension; one

beam by tension in steel and one beam by tension in steel and/or in bond. The results of this test are available but unpublished.

The present active interest in Reinforced Brick Masonry, however, appears to date from the year 1922, when Mr. A. Brebner, Under Secretary, Government of India, Public Works Department, published in two volumes a classic report entitled "Notes on Reinforced Brickwork", in which it is reported that reinforced brick masonry has had wide use in India, several of the local Indian Governments have adopted it in the construction of public buildings, as well as in private dwellings for officials. In all, nearly three million square feet of reinforced brick masonry construction are said to have been built in India during the three years prior to 1922.

Mr. Brebner's report includes the results of several hundred tests conducted on many types of slabs, beams, cantilevers, etc. As a basis for comparison, many tests on similar reinforced concrete specimens were made, from the results of which it is concluded in the report, that the theory of reinforced concrete design may properly be applied to reinforced brick masonry design. The same conclusion has since been reached by a number of investigators in the United States and other countries.

A pamphlet, published in 1923 by Eugene S. Powers, describes reinforced brick masonry under the name of "Adhesive Construction in Brickwork". In this pamphlet some of the early uses of this type of construction are described and data obtained by Mr. Powers from personal tests are reported.

In 1924, the Royal Engineers Journal (England) published an article by Captain T. W. D. Miller, R. E., written from notes made on the work in connection with the erection of officers' quarters at Cheklala, India, in which is described the design and construction of roof slabs of reinforced brick masonry. The roofs were originally designed as reinforced concrete joist and tile construction, and it is noted that the design for the reinforced brick masonry slabs was based on reinforced concrete design formulas.

Since 1920, Japan has been using reinforced brick masonry in governmental construction. In more recent years, it has also been used in private works. Dr. Shigeyuki Kanamori, civil engineer with the Japanese government, reports great success with R-B-M construction. It is used in buildings and all types of engineering structures, such as retaining walls, culverts, bridges, sea walls, gate towers, levees and quays. Reinforced brickwork, it is claimed, combines the advantages of reinforced concrete with the durability, fireproof quality, good appearance and simple, fast construction of brickwork, with a considerable saving of time and construction expense.

Research sponsored by the Brick Manufacturers Association of America and continued by the Structural Clay Products Institute has contributed much valuable material to the literature on the subject of Reinforced Brick Masonry. Since 1924, numerous field and laboratory tests have been made on reinforced brick masonry beams, slabs and columns and on full size structures. In 1932, the Reinforced Brick Masonry Research Board was organized. This Board is composed of prominent professors in engineering schools, outstanding consulting engineers and other authorities on structural engineering. A comprehensive research program has been outlined, and the Board has done effective work in directing and coordinating tests on materials, mortars, plain and reinforced brick masonry. The reinforced brick masonry units tested have included beams, slabs, walls and columns. Some of the laboratories which have cooperated with the Board or have conducted independent R-B-M tests include the National Bureau of Standards, University of California, Case School of Applied Science, California Institute of Technology, Columbia University, Cornell University, University of Detroit, University of Illinois, Massachusetts Institute of Technology, University of Missouri, University of Pennsylvania, Purdue University, Rensselaer Polytechnic Institute, Southern Methodist University, Virginia Polytechnic Institute, University of Washington and University of Wisconsin. The complete results of many of the laboratory and field tests,

reported above, have been published in various magazines, bulletins and research papers, a list of which will be found in the appendix.

The Long Beach, California earthquake of March 10th, 1933, and the so-called earthquake law which was enacted by the California State legislature in May of the same year, greatly stimulated the use of reinforced brick masonry on the Pacific Coast. During the past six years, hundreds of reinforced brick masonry buildings have been constructed, including schools, motion picture studios, commercial buildings, and dwellings. Extensive research has been carried on and construction practices have been developed. Probably the most important of the latter is the grouting method of construction. This method consists of filling all interior joints, including the inner portions of the head and bed joints of the facing wythes, with a freely flowing grout. No masonry headers are used. It has been found that grouting practically eliminates the danger of partially filled mortar joints with the resultant decrease of masonry strength and the grout binds the entire assemblage together into a solid mass. In the California area, grouting interior joints in reinforced brick masonry has completely replaced the methods of filling formerly employed with mortars of brick laying consistency and this method is spreading to other areas for the construction of plain as well as reinforced brick masonry.

## 903. ADAPTABILITY OF REINFORCED BRICK MASONRY TO MODERN BUILDING CONSTRUCTION

From the experiences obtained in the construction of the numerous structures of reinforced brick masonry built during recent years, particularly in Southern California, it may be said that the construction of reinforced brick masonry offers no practical difficulty to the experienced mason contractor or bricklayer nor is it excessive in cost. Under certain conditions it will be found less costly than other commonly used types of construction. When correctly designed, the introduction of reinforcing bars into brick masonry utilizes to the greatest possible extent the full crushing strength of the individual bricks, and of the mortar, and the combination of reinforcing steel and plain brick masonry results in a construction in which both materials are effectively combined to their mutual advantage.

In common with reinforced concrete, reinforced brick masonry develops a high structural efficiency in a member through the combination of the reinforcing steel with the plain masonry section. Plain brickwork possesses a relatively low tensile strength, but its compressive strength is high, particularly when the brick are laid in a strong mortar.

Steel is particularly well adapted to resisting tensile stresses. The small area and number of bars ordinarily required in a beam or slab design is such that no difficulty is ordinarily experienced in suitably and efficiently introducing the reinforcing steel into the mortar joints.

An outstanding advantage of reinforced brick masonry is that frequently forms are not required and, if required, they need not be watertight. For slabs, the usual shoring is used, but the deck can be built up of light framing lumber covered either with square edge boards (not necessarily placed tight) or with light removable steel sheets, plywood, Sisalkraft paper, Masonite or other similar coverings. For the soffit of lintels, beams and girders, a single board of suitable thickness, not necessarily the full width of the member, is sufficient. Vertical surfaces, such as beam sides, walls and columns need no forming at all. Curved walls are easily built without the use of intricate forms. In general, the forms used are of the least expensive kind.

The weight of reinforced brick masonry is generally less than that of reinforced concrete, and ranges from 117 pounds to 135 pounds per cubic foot as compared with 145 pounds to 150 pounds for concrete, a saving in weight of approximately 20%. This saving in weight is of real economic importance and should not be overlooked.

The architectural possibilities of reinforced brick masonry are many-fold. Not only can the same attractive facing brick materials and style of bonding now in common use be employed, but the desired surface texture, colors and pattern-work of the exposed surface can be obtained as an integral part of the structural member itself, thus eliminating the extra cost of supporting members, such as shelf angles, etc.

The first cost of reinforced brick masonry is comparable to other similar competitive forms of masonry construction. Contractors who have had no experience with this type of construction are apt to estimate the labor required as considerably greater than for plain masonry. However, in localities where reinforced brick masonry is in common use, it has been found that the production of the masons is not impaired by the reinforcing steel and work is being estimated accordingly. As with plain brick masonry maintenance costs are reduced to a minimum.

Fire tests made on this type of construction prove the ability of reinforced brickwork to effectively resist the attack of devasting fires for long periods of time. Plain brickwork is the actuarial basis for scheduled minimum fire insurance rates; therefore, it seems reasonable and logical to suggest that reinforced brick masonry walls should be entitled to a preferred fire insurance rating, and that the minimum wall thicknesses, now stipulated for plain brick masonry, may properly be reduced as in the case of reinforced concrete.

A field test of the resistance to earthquake of reinforced brick masonry occurred during the California earthquake of October 2, 1933, near Los Angeles. After this disturbance, 61 reinforced brick buildings, constructed of "Groutlock" brick, manufactured by the Simons Brick Company of Los Angeles, were inspected and it was found that "in no instance was there the slightest damage to any of the walls or chimneys."

In proportioning a reinforced brick masonry member, it is convenient to adjust its over-all dimensions to some multiple or function of the unit brick size and to the desired maximum or minimum thickness of mortar joints. This arrangement proves to be a decided advantage in preventing misplacement of reinforcing steel and in definitely fixing its position in the masonry. Dimensions of structural members are usually fixed more or less arbitrarily by the designer, except in those instances where headroom or side clearance is a critical requirement. However, since the thickness of the mortar joints generally offers some latitude, the designer will find it possible to adjust the dimension of an R-B-M member within quite close limits. Numerous special shapes have been developed for use especially in grouted masonry which aid in controlling over-all dimensions.

A knowledge of standard or local brick sizes and shapes is essential, as is also familiarity with the many possible bond combinations of brick laid flatwise, on edge and on end. This knowledge of brick masonry and bricklaying technique is easily acquired and should present no difficulties to the practicing architect or engineer.

Reinforced brick masonry has been used successfully in buildings of many types and in practically every type of engineering structures, such as abutments, athletic stadiums, bridges, caissons, culverts, chimneys, conduits, dams, levees, piers, reservoirs, retaining walls, sea walls, sewers and sewage treatment plants, silos, storage bins and tanks, subways, trestles, tunnels, etc. The most common use for R-B-M in buildings has been for foundations, columns, walls, beams and lintels, while other uses include piers, floor slabs, balconies, stairways and parapets. Paving brick have been used, as one would naturally suppose, in reinforced brick pavements. Most of these have been test installations, but the success of these experiments indicates the vast possibilities for reinforced brick paving on highways, bridge slabs, driveways and courts. Another field for paving brick, along with sewer brick and low density building brick, is in structures subject to the abrasive and other disintegrating actions of water. Here the brick may be used for the entire structure or as a protective facing and form for a concrete

core in wing walls, piers, side walls and floors of sewers, conduits and outlet structures of dams, reservoirs, etc.

While reinforced brick masonry may be adapted to practically any type of modern structure, its cost as compared to competing types of construction will, as in other materials, tend to concentrate its use in structures in which substantial economies can be effected. One of the most outstanding of such constructions are vertical walls requiring a high resistance to lateral forces. Such walls as retaining walls, foundation or building walls designed to resist earthquake shock or excessive wind pressures, can usually be constructed at less cost than other comparable types of construction.

Frequently, architectural considerations will also dictate the choice of a material from two or more equally satisfactory, from a structural standpoint, and obtainable at approximately the same cost.

## MATERIALS

### 904. GENERAL

Principal material constituents of Reinforced Brick Masonry are: (a) Brick; (b) Mortar and Grout; (c) Reinforcing steel. The durability and strength of reinforced brick masonry depends on the quality of the materials used, design and workmanship. Best results will be obtained by a proper selection of well-burned brick; careful proportioning and mixing of the mortar and grout; proper wetting of the brick before laying; complete embedment of the reinforcing steel and completely filled joints.

The physical properties of building brick, mortar and brick masonry have been presented in Chapters 1, 2 and 3. In this chapter, the properties will be discussed with relation to their utility or effect in reinforced brick masonry.

### 905. BRICK

#### (a) SIZE

The type of brick most commonly used in reinforced brick masonry are building, sewer and paving brick. For convenience, the standard sizes are listed herewith.

| Type of Brick | Depth | Width | Length |
|---|---|---|---|
| Standard building | 2¼″ | 3¾″ | 8″ |
| Smooth-face building | 2¼″ | 3⅞″ | 8″ |
| No. 1 Sewer | 2¼″ | 3¾″ | 8″ |
| No. 2 Sewer | 2½″ | 4″ | 8½″ |
| No. 3 Sewer | 3″ | 4″ | 8½″ |
| No. 4 Sewer | 3½″ | 4″ | 8½″ |
| Vertical Fiber Lug paver | 2½″ | 4″ | 8½″ |
| Vertical Fiber Lug paver | 3″ | 4″ | 8½″ |
| Vertical Fiber Lug paver | 3½″ | 4″ | 8½″ |
| Repressed Lug paver | 4″ | 3½″ | 8½″ |

Variations from these sizes will be found in many localities. There are several special shapes of building brick for use in reinforced brick masonry. One of the better known of these units, the "Groutlock" brick, has been widely used on the Pacific Coast in R-B-M construction. Fire brick and glazed brick are produced in a large number of standard sizes and shapes. Standard sizes of glazed brick and tile may be obtained from the handbook prepared by the Glazed Brick and Tile Institute and included in such

publications as Sweet's Catalogue. It is recommended that before an actual design is made in which special brick are to be used, the designer ascertain from the manufacturer or dealer, the average size and the tolerance of the several types of brick intended to be used.

### (b) COMPRESSIVE STRENGTH

From the standpoint of compressive strength, only well-burned brick having a minimum compressive strength (flatwise, on edge, or on end) of 2500 pounds per square inch should be used in Reinforced Brick Masonry.

In column, pier and wall construction, the flat compressive strength of the brick is considered most important because the brick are laid in the wall with the load acting, in general, at right angles to their flat faces. The flat compressive strength is also important when the brick are laid in a lintel or beam as soldiers; on edge as rowlock, or when laid flat in a column or pier. In reinforced brick masonry beams and slabs, however, while the direction of the applied load may be the same, the flexural action within the beam or slab results in stresses applied at right angles to the edge of brick (when laid as headers) or against the ends of brick (when laid as stretchers), therefore these two properties of a brick, namely compressive strength on edge and on end, are then equally, if not more important than the flatwise strength.

The flatwise strength of many brick manufactured in this country is greater than the edgewise strength; therefore, the edgewise strength requirement may often control. In thin slabs, one brick in thickness, laid flatwise or on edge, the end compressive strength of the brick should be determined.

### (c) TRANSVERSE STRENGTH

The transverse strength of an individual brick is generally more important in slabs than in beams. Therefore, the use of a brick having a minimum modulus of rupture of 500 lb. per sq. inch appears highly desirable in slab construction. Limited data seem to indicate a rather close relation between tensile strength of brick and modulus of rupture, ranging from 30% to 40%. Therefore, it would appear that the modulus of rupture value may be taken as a measure of the probable tensile strength of a brick. However, present data indicate that tensile strength of brick is not an important factor and need not be a specification requirement in Reinforced Brick Masonry.

### (d) SHEARING STRENGTH

If brick are well-burned, it is quite unlikely that their shearing strength will be an important consideration, since the shearing strength of brick ranges from three to six times the tensile strength and the likelihood of a brick failing in direct shear is quite remote.

### (e) ABSORPTION

The bond value between brick and any mortar appears to be related to the rate of absorption and, in a smaller degree, to surface roughness of the brick. Tests show that all other things being equal, the best mortar bond is obtained by using brick having an absorption when laid sufficient to prevent the brick from "floating" on the mortar bed but not over 20 grams in a minute when laid flatwise in $\frac{1}{8}$" of water. For grouted masonry the rate of absorption may safely be raised to 30 grams per minute.

High absorption brick are satisfactory for the construction of reinforced brick masonry, provided this rate of absorption (or penetrability as it is frequently termed) of the brick when laid has been reduced to within the above limits by adequate wetting. The rate of absorption of brick depends upon the raw materials and methods of forming and burning used in manufacture and should be controlled by wetting the brick before

laying rather than by an attempt to specify fixed normal rates of absorption which may not be obtainable in the products of certain localities.

### (f) SURFACE ROUGHNESS

The roughness of surface of a brick is sometimes important in reinforced brick masonry, since the horizontal and vertical shearing forces cause critical bond stresses, particularly in beams. It has been observed that beams and slabs built of brick having rough wire cut bed faces in contact with the mortar, develop somewhat higher resistance to bending and web stresses than do similar beams or slabs in which relatively smooth brick have been used. It has also been noted that brick containing a multiplicity of small cored holes at right angles to the bed face may likewise aid in developing higher beams resistance.

### (g) WEATHER RESISTANCE

To obtain weather resistance in brickwork, a well-burned brick should be used in all locations exposed to the weather. The tentative standard A.S.T.M. Specifications for Building Brick (C62-37T) have been written primarily to classify brick according to resistance to weathering. The combined properties of brick which are considered as best indications of weather resistance are strength, absorption and saturation coefficient (C/B ratio). All brick meeting the qualifications for the grade to resist severe weathering will undoubtedly give satisfactory service. There are, however, many makes of brick which may not entirely meet these specifications, but from actual service as well as laboratory freezing-and-thawing tests have proven that they will resist the most severe weather conditions. It is difficult to write a general specification without some exceptions to the rule. The designer is reminded, therefore, that the demonstrated ability of a particular kind of brick to withstand seasonal changes and exposure over a long period of years under actual climatic condition, is the best indication of its weathering qualities. Field observations, made in many parts of the country, show that properly built walls of well-burned brick are weather resistant.

## 906. MORTAR MATERIALS

All mortar materials should be of first quality and should conform to standard specifications.

### (a) PORTLAND CEMENT

Portland cement should conform to the requirements of A.S.T.M. Specifications, Serial Designation C9-38. High-early-strength portland cement should conform to the requirements of A.S.T.M. Specifications, Serial Designation C74-38.

### (b) LIME

Lime may be purchased as: (a) Lump quick lime; (b) Ground quick lime; (c) Hydrated Lime; (d) Lime Putty.

Lime suitable for use in mortar for reinforced brick masonry may be any one of the several kinds, provided it conforms to the current A.S.T.M. Specifications for Quicklime or Hydrated Lime for Structural Purposes. It is preferable that lime be used in the form of putty, well-aged. The slaking of quicklime and preparation of lime putty should conform to the suggestions contained in the appendix to A.S.T.M. Specifications for Quicklime for Structural Purposes (C5-26).

### (c) SAND

Natural sand is understood to be the product of the natural disintegration of siliceous or calcareous rock. Manufactured sand is usually qualified in name as from

crushed stone, quarry sand, etc., as distinguished from screenings, gravel, etc. Too much care cannot be exercised in selecting a good grade of sand for mortar. It should be clean and composed of quartz grains or other hard material selected for its quality and grading. The use of uniformly grained sand is dangerous and should be avoided. All sand used for mortar or grout in R-B-M should be clean, hard, well-graded, and free from objectionable organic matter or other impurities. Reference is made to Chapter 2 for recommendations on grading and deleterious substances.

### (d) WATER

Only clean water, free from acids, alkalies, oils and other impurities, should be used. Salt water should never be used.

## 907. REINFORCING STEEL

Bar reinforcement should conform to the requirements of the current A.S.T.M. Specifications for Billet-Steel or Rail-Steel Reinforcement Bars.

Welded wire fabric or cold-drawn wire for reinforcement should conform to the requirements of the current A.S.T.M. Specifications for Cold-Drawn Steel Wire for Reinforcement.

## 908. MORTAR AND GROUT

As indicated in Chapter 2 the mortar recommended for general use in reinforced masonry consists of one part cement, .25 part lime putty and three parts sand, by volume. These proportions may be varied, however, for specific work.

Tests at Virginia Polytechnic Institute "A Comparison of the Performance Characteristics of Reinforced Brick Masonry Slabs and Reinforced Concrete Slabs" indicate that reinforced brick slabs, one brick on edge thick ($3\frac{3}{4}''$) constructed with a 1:1:6 mortar (one part cement, one part lime, six parts sand, by volume) have approximately the same characteristics, including strength as reinforced concrete slabs of the same depth (effective and over-all) and reinforcement using 2000 lbs. per sq. in. concrete. Since relatively high strengths are usually required in R-B-M, low strength mortars can rarely be used efficiently and based upon the present available data, mortars in which the proportion of lime by volume exceeds the cement are not recommended.

When mortars of lower strength than the $1:\frac{1}{4}:3$ mixture recommended for general use are used, allowable working stresses should be reduced. Definite data as to the amount of reduction, particularly of the allowable bond and shearing stresses are lacking, however, a reduction of all working stress in proportion to the cube root of the mortar 7-day cube strength is suggested.

In the development of grouted reinforced brick masonry in California, grout was first produced by adding sufficient water to a cement sand mixture, consisting of one part cement to three to five parts sand, by volume, to give the mixture a freely flowing consistency. It has been found, however, that additions of lime of from 10 to 25 per cent of the cement by volume, add to the fluidity of the grout and tend to reduce segregation of the ingredients without materially reducing its strength. For general use, a grout consisting of one part cement, .15 to .25 parts lime, three parts sand, by volume, is recommended. Sands containing a high percentage of fines (3 to 5% passing No. 200 sieve) will produce satisfactory grout with the addition of the smaller amounts of lime; with coarser sands more lime is required.

Tests have been conducted on reinforced grouted walls in which the grout was obtained by adding water and cement (2 lb. of cement with each quart of water) to the mortar in an attempt to keep the water cement ratio of the mortar and grout constant. The results of these tests (to be published by National Bureau of Standards) indicate a high bond strength of the grout; however, the data are too limited to justify definite recommendations at this time.

# STRENGTH AND WORKING STRESSES OF R-B-M

## 909. LABORATORY TESTS

Numerous laboratory tests have been conducted on reinforced brick masonry beams, slabs and columns to obtain design data and information on which to base working stresses. The results included in the following tables are typical of the findings of other investigators, many of whose reports are listed in the appended Bibliography.

Table 26 is a summary of tests conducted at the Engineering Materials Laboratory of the University of California during the period October 1929 to December 1930. This table is included in the Report to Members of Committee C-12 of the American Society for Testing Materials by Raymond E. Davis, Chairman, issued under date of May 31, 1933, and indicates the bond and tensile (modulus of rupture) strengths that may be obtained in plain masonry without reinforcement.

### TABLE 26

### RESULTS OF TESTS ON MASONRY (1929 SERIES)

| | Strength at Age of 6 Months Lb./sq. in. | | | |
|---|---|---|---|---|
| | Mortar (1:2:6) | Mortar (1:1:6) | Mortar (1:½:4½) | Average |
| Tensile strength of mortar briquets | 215 | 250 | 332 | 266 |
| Compressive strength of 2 by 4-in. mortar cylinders | 1590 | 1700 | 2430 | 1910 |
| Tensile strength in bond of masonry | | | | |
| Brick A | 27 | 33 | 81 | 47 |
| Brick B | 30 | 28 | 87 | 48 |
| Brick C | 64 | 49 | 67 | 60 |
| Average | 40 | 37 | 78 | 52 |
| Modulus of rupture of 4 by 8 by 24-in. masonry beams | | | | |
| Brick A | 136 | 96 | 178 | 137 |
| Brick B | 67 | 115 | 105 | 96 |
| Brick C | 129 | 201 | 165 | 165 |
| Average | 111 | 137 | 150 | 133 |
| Modulus of rupture of 12 by 12 by 54-in. masonry beams | | | | |
| Brick A | ... | 82 | 95 | ... |
| Bond of masonry with ⅝ in. round plain steel bar | | | | |
| Brick A | 234 | 317 | 365 | 305 |
| Shear (across 12 by 12 in. masonry pier) | | | | |
| Brick A | ... | 86[a] | 130[a] | ... |

a—Tested at age of 8 months.

Mortar proportions by volume, cement, lime putty, sand.

Masonry specimens laid wet, sprinkled twice daily for 14 days, then stored dry until test.

Characteristics of brick were as follows:

| Brick | Type | Absorption per cent | Compressive Strength Lb./sq. in. | Modulus of Rupture Lb./sq. in. |
|---|---|---|---|---|
| A | Wire-Cut | 12.0 | 3040 | 586 |
| B | Sand-Mold | 13.6 | 3870 | 598 |
| C | Face | 8.6 | 7710 | 1490 |

Tables 27, 28, 29 and 30 are summaries of tests conducted at the National Bureau of Standards and reported in Research Paper No. 504. The mortar used consisted of one part portland cement to three parts sand by volume, with an addition of hydrated lime equal to 15% of the volume of cement.

## TABLE 27

## SUMMARY OF BEAM TEST RESULTS

| Beams | Value of j | Shearing Stress $v = \frac{V}{bjd}$ lb. per sq. in. | | |
|---|---|---|---|---|
| | | Average | Maximum | Minimum |
| CA | .85 | 159 | 171 | 153 |
| CB | .85 | 131 | 140 | 123 |
| CC | .78 | 94 | 105 | 87 |
| PA | .83 | 97 | 111 | 89 |
| PB | .83 | 92 | 105 | 80 |
| PC | .77 | 65 | 81 | 48 |

NOTE: $j = 1 - \frac{k}{3}$ when k is the average of experimentally determined values of rates of depth of neutral axis to effective depth. Tests indicate that the maximum shearing stresses were in the same order as the proportion of bricks laid with staggered vertical joints.

## TABLE 28

## SUMMARY OF PIER TESTS

| | Secant Modulus of Elasticity to 250 Lbs./sq. in. | | | Compressive Strength Lbs./sq. in. | | |
|---|---|---|---|---|---|---|
| Piers | Average | Maximum | Minimum | Average | Maximum | Minimum |
| CA | 2,480,000 | 2,690,000 | 2,340,000 | 1,856 | 1,955 | 1,735 |
| CB | 2,210,000 | 2,380,000 | 2,120,000 | 2,025 | 2,458 | 1,661 |
| CC | 1,180,000 | 1,570,000 | 960,000 | 1,021 | 1,235 | 810 |
| PA | 1,540,000 | 1,800,000 | 1,050,000 | 1,688 | 2,039 | 1,438 |
| PB | 1,070,000 | 1,220,000 | 980,000 | 1,389 | 1,535 | 1,210 |
| PC | 870,000 | 1,070,000 | 740,000 | 1,202 | 1,423 | 1,070 |

NOTE: Piers A and B have some bricks on end and Piers C have all bricks laid flatwise. Height to thickness ratio of all piers, approximately 3.

## TABLE 29

## CHARACTERISTICS OF BRICK

| Kind of Brick | Average Compressive Strength Lb./sq. in. | | Average Absorption % | |
|---|---|---|---|---|
| | Flatwise | Edgewise | 48 hr. cold | 5 hr. boil |
| Chicago | 3910 | 4280 | 10.8 | 14.7 |
| Philadelphia | 4510 | 5240 | 12.7 | 16.1 |

Table 31 gives the results of tests on reinforced brick masonry beams, conducted at the University of Wisconsin and reported in a paper presented by M. O. Withey at the 1933 annual meeting of the A.S.T.M.

## TABLE 30

## TESTS ON REINFORCED BRICK MASONRY BEAMS

| Beam | Reinforcement | | Stresses in Lbs./Sq. In. Calculated from Maximum Moment | | | | Kind of Failure |
|---|---|---|---|---|---|---|---|
| | Longitudinal percent | Shear | Steel $f_s$ | Brick $f_b$ | Shear V | Bond U | |
| C1 | .57 | One ¼″ Z at 6″ | 56900 | 1330 | 95 | 168 | T |
| C2 | .56 | One ¼″ Z at 6″ | 55600 | 1270 | 96 | 161 | T |
| C4 | .61 | None | 57000 | 1370 | 96 | 168 | T |
| C5 | .98 | Two ¼″ Z at 3″ | 53300 | 1740 | 148 | 202 | C |
| C6 | .93 | Two ¼″ Z at 3″ | 50300 | 1580 | 140 | 189 | T |
| C9 | 1.23 | One ⅜″ Z at 3″ | 54100 | 2040 | 208 | 210 | T DT |
| C3 | .62 | None | 53100 | 1290 | 91 | 156 | DT |
| C7 | 1.33 | One ⅜″ V at 8″ | 36900 | 1470 | 140 | 143 | DT |
| C8 | 1.24 | One ⅜″ Z at 3″ | 36800 | 1400 | 133 | 143 | DT |
| W3 | .62 | One ¼″ Z at 6″ | 56500 | 1610 | 97 | 160 | T |
| W4 | 1.00 | Two ¼″ Z at 6″ | 48100 | 1840 | 144 | 181 | T |
| W5 | 1.04 | Two ¼″ Z at 4″ | 49400 | 1920 | 143 | 184 | T |
| W6 | 1.02 | Two ¼″ Z at 4″ | 49500 | 1910 | 148 | 187 | T |
| W7 | 1.41 | One ⅜″ Z at 3″ | 52800 | 2490 | 212 | 201 | C |
| W8 | 2.31 | One ⅜″ Z at 3″ | 45300 | 2970 | 294 | 226 | C |
| W1 | .66 | None | 57800 | 1710 | 100 | 164 | T DT |
| W2 | .70 | None | 57800 | 1770 | 109 | 172 | DT |
| S1 | .66 | None | 56000 | 1360 | 98 | 167 | T |
| S2 | .66 | None | 65700 | 1600 | 115 | 194 | T |
| S3 | 1.15 | None | 47200 | 1640 | 146 | 186 | T |
| S4 | 1.29 | One ⅜″ Z at 3″ | 54000 | 2020 | 203 | 209 | T |
| S5 | 1.06 | Two ¼″ Z at 4″ | 46800 | 1530 | 142 | 182 | T |
| S6 | 1.03 | Two ¼″ Z at 6″ | 48800 | 1580 | 148 | 191 | T |
| S8 | 1.30 | Two ¼″ Z at 4″ | 57600 | 2170 | 219 | 222 | T |
| S7 | 2.22 | One ⅜″ Z at 3″ | 37800 | 2050 | 243 | 188 | DT |

C, compression; T, tension; DT, diagonal tension.

NOTE: Compressive strength of C prisms was 2690, of W prism was 2870, and S prism was 2090. Mortar was three parts Cement, one part Hydrated Lime and twelve parts of Sand — All by Weight.

## TABLE 31

## RESULTS OF PULL-OUT TESTS

| Kind of Brick | Storage | No. of Specimens | Average Days | Average Bond Strength Lb./sq. in. |
|---|---|---|---|---|
| Chicago | Dry | 5 | 32–48 | 880 |
| Philadelphia | Dry | 4 | 32–48 | 950 |
| Chicago | Damp | 5 | 41–57 | 870 |
| Philadelphia | Damp | 5 | 41–57 | 920 |

NOTE: Specimens were deformed steel bars ½-inch square, embedded about 8 inches in 8x8 inch square brick masonry piers. All brick were wet at time of laying.

As will be noted, only three beams failed in compression. The compressive strengths of the brick masonry in these three beams, aged 14 days, were 1740, 2490 and 2970 lbs./sq. in.

Test prisms, made of the same brick and mortar used in the two beams showing the higher values, had an average compressive strength of 2870 lb./sq. in. These prisms were approximately 8″ x 8″ x 25″. The mortar was composed of one part portland cement, one-third part hydrated lime and four parts sand by weight. Three kinds of brick were used, with three to five prisms made of each kind. The following table shows related test data:

## TESTS ON R-B-M BEAMS AT AGE OF 14 DAYS

| Kind of Brick | Average Compressive Strength of Brick Unit | | Average of Pier or Prisms | | |
|---|---|---|---|---|---|
| | Flat Lb./Sq. In. | Edge Lb./Sq. In. | End | Compression | Modulus of Elasticity |
| Chicago | 2586 | 2390 | 5550 | 2690 | 2,400,000 |
| Waupaca | 12500 | 5860 | 5900 | 2870 | 3,500,000 |
| Streator | 9195 | 4034 | 3970 | 2090 | 2,100,000 |

Inasmuch as the brick in the prisms were tested on end, that may account for the apparent correlation between the average strength of piers and the average strength of the brick in end compression. It will be noted that the compressive strength of the brick prisms or piers was approximately one-half the average strength of the brick in end compression.

Tests made in 1933 at the University of Wisconsin, by Professor M. O. Withey on reinforced brick masonry columns (See A.S.T.M. Prov. Vol. 34, Part II), showed compressive strengths of 8 x 8 x 16-inch prisms made with Waupaca brick laid flat in the same 1:1/3:4 mortar as averaging 3675 lb./sq. in. at an age of four weeks. With everything else remaining constant, except the mortar mix, which was changed to 1:1/3:2½ the average compressive strength of prisms was 4521 lb./sq. in.

Tests previously referred to were on Reinforced Brick Masonry constructed with mortar of brick laying consistency only. Recent tests on grouted reinforced brick masonry indicate that somewhat higher strengths may be obtained with this type of construction. The following is a summary of the "Report on Tests of Bond Strength of Reinforcing Steel in Groutlock Brick" submitted by Raymond G. Osborne, Bureau of Tests and Inspection, Los Angeles, California, under date of August 27, 1938.

"The two test walls under investigation, designated as "B" and "C", were each some 12′-6″ in length, 3′-0″ in height, and 8″ thickness. They were constructed of two parallel brickwork sections enclosing a section of grout approximately 1″ thick in which were embedded ⅜″ deformed round steel reinforcement bars spaced 18″ on centers vertically, and two horizontal bars (below the range of investigation). The brick were of the type known as GROUTLOCK, made by the soft mud process, having all around open pore rough surfaces.

We are informed that the walls were laid on May 11th, 1938, and that the construction conformed to the specifications for brick, mortar, and grout embodied in the Los Angeles County (Uniform) Building Code, the grout having the proportions of one to three and the mortar of one to three with addition of 15 per cent of lime putty by volume to the cement. A free flowing grout was used (having a slump of some 11.5″), poured into and completely filling the center joint at the completion of each course, and trowel stirred immediately after pouring. The vertical steel (on which all tests were made) was braced to reduce swaying during construction, though throughout the laying of wall "C", the bars were violently shaken when each course had been completed."

"Having determined by the preliminary work on June 25th that an embedment in the neighborhood of 5″ was more than adequate to completely develop the strength of the steel, the test specimens (chosen at random from the vertical steel bars projecting from the top of the walls) were exposed by means of holes cut through the walls, and cut off in such fashion as to leave sections of steel embedded between 3.0″ and 3.68″ in effective length. On the basis of the theoretical area of 1.18 square inches per lineal inch of ⅜″ smooth round rod assumed, the areas in bond ranged between 3.54 and 4.34 square inches.

In order to develop an adequate and accurately determinable pull on the bars, a beam was contrived to rest at one end on a firmly fixed knife edge and at the other on a knife edge which rested on the ram of an hydraulic jack, while at the midpoint a third knife edge supported a saddle which, in turn, was firmly clamped to the free end of the bar to be tested. This system exerted a force in tension on the bar equal to twice the force exerted by the jack, and, since the jack was equipped with an accurate gauge for measuring the pressure in the ram chamber (calibrated before and after the experiments on an Olsen Testing Machine of known accuracy), the actual pull on the bar was easily determined. As an additional check the elongation of the bars under stress were several times measured by means of Hugenberger Extentiometers of Mr. Paul E. Jeffers, structural engineer, and the results found to confirm the indicated force. In order that the wall itself might not be affected by the exerted force, the whole equipment was supported on a 1″ thick steel plate which rested on the top of the wall and had a 1½″ hole cut for the passage of the bar.

The slip of the bars under stress was determined by means of a micrometer dial gauge firmly clamped to the wall in such a fashion that its actuating arm rested against the lower face of that section of the rod under test.

The pull on the bars was increased in increments approximating 320 pounds each, and after each increase the pressure sustained until all evidenced movement at this pressure had ceased; this total slip was then recorded and an additional increment applied. At the completion of the tests the stress on the steel-grout bond per square inch of area embedded required to produce movements of 0.0001″ (first observed movement), 0.0005″, 0.00020″, 0.0050″, and 0.0100″, and the ultimate pull-out was computed; and the results compiled in the attached Table No. 32.

On the whole the evidenced bond values were exceptionally uniform and much higher than had been anticipated. The extreme stress to which the bond was subjected without appreciable movement ranged between 532 and 697 pounds per square inch of area in the five specimens tested, while that at which the first observed slip occurred

ranged from 625 to 775 pounds (n.b. — all values computed on the basis of the theoretical area of 1.18 square inches per lineal inch of embedment of ⅜" smooth round rod).

It is interesting to observe that the ultimate pull-out of the specimens occurred in no instance until the pull on the rods had well exceeded the yield point of the steel (in the neighborhood of 50,000 pounds per square inch) and, hence, may be regarded as having occurred progressively. The required stress on the bond amounted to between 1550 and 1842 pounds per square inch.

One of the primary purposes of the test program was to determine the effect of strong agitation on the reinforcement rods during construction (embodied in the construction of wall "C"). The results of the tests showed no appreciable divergence in trend, and as a result the data from both walls has been combined in the summary, though the behavior of the individual specimens may be identified as between the two walls by their designations in Table No. 32."

**TABLE 32**

**TEST DATA**

| Slip of Bar, In. | Bond Stress in Lbs. per Sq. In. of Embedment | | | | | |
|---|---|---|---|---|---|---|
| | B-2 | B-6 | B-7 | C-5 | C-6 | Average of 5 |
| 0.0001 | 720 | 625 | 763 | 664 | 775 | 710 |
| 0.0005 | 955 | 905 | 907 | 766 | 1170 | 942 |
| 0.0010 | 1106 | 1036 | 1050 | 901 | 1250 | 1068 |
| 0.0020 | 1230 | 1239 | 1205 | 1165 | 1274 | 1222 |
| 0.0050 | 1367 | 1308 | 1311 | 1270 | 1348 | 1321 |
| 0.0100 | 1425 | 1365 | 1374 | 1286 | 1420 | 1373 |
| Ultimate | 1750 | 1550 | 1842 | 1605 | 1775 | 1704 |
| Note: Approximate yield of steel | 1460 | 1460 | 1370 | 1190 | 1250 | .... |

The Smith-Emery Company of Los Angeles reported the results of a test on a grouted reinforced brick beam under date of December 14, 1937. Data on the beam tested are given as follows:

Width of beam ..... 12.5 inches
Length of beam ..... 14.0 feet
Height of beam ..... 18.0 inches

**Proportions:**

Mortar — 1:3 (15% lime putty of cement content)
Grout — 1:3

**Reinforcing steel:**

Four 1-inch round bars hooked at the end placed 15.5 inches down from the top and 2 inches in from the side. ½-inch stirrups. 12-inch centers. Average (5 specimens) compressive strength of brick 2879 lb./sq. in.

This beam was tested at the age of 167 days on a clear span of 13 feet. The maximum load concentrated at the center was 40,000 lbs., which with the dead load (masonry figured at 125 lbs. per cu. ft.) produced a maximum bending moment of 1,609,512 inch pounds. Maximum stresses computed at maximum load, were 2276 lbs. per sq. in. com-

pression in masonry and 40,420 lb. per sq. in. tension in steel. The average calculated modulus of elasticity of the masonry for all loads was 1,550,986 lb. per sq. in., and averaged for loads producing stresses in the masonry of 500 to 1400 lbs. per sq. in. was 1,532,933 lb. per sq. in. The modulus of elasticity of the steel was found to be approximately 29,500,000 lbs. per sq. in. giving a value of "n" slightly over 19. After the beam test, two cores approximately 8″ in diameter x 12″ high were drilled from the beam and tested in compression. The ultimate strength of these cores were 2568 and 2931 lbs. per sq. inch respectively.

## 910. COMPRESSION TEST SPECIMENS

Numerous investigators have attempted to correlate the bond, shearing and compressive strength in bending of R-B-M with the ultimate compressive strength of brick masonry prisms. A form of prism which has been used quite extensively is approximately 8 inches square by 25 inches long, built in horizontal position with unselected brick laid as stretchers in running bond, two brick wide, three brick long and three courses high with ½″ joints. The brick are laid flat on a smooth surface, with the outside edge of the ends buttered. The extreme ends of the longitudinal vertical joint are filled with mortar by buttering the inside edge at the ends of the first and last brick. The inside vertical joints are then filled with grout. A flat bed of mortar is spread for the second course and the brick shoved into place. The mortar and grout are placed as described for the first course. The third course is similar to the second. The mortar joints are tooled, preferably with a round jointer.

In the event that plastic mortar joints only are used in R-B-M, the jointing in the test specimen is also made with mortar of the same consistency and with the quality of workmanship recommended for R-B-M construction.

When these prisms are tested, the brick of which they are constructed are in end compression and results might be expected to be related to the strength of structures in which the brick were similarly placed. In many types of wall construction, however, the compressive force is applied to the flat side of the brick and for such construction, prisms in which the brick are so placed that they will be in flatwise compression would appear to give results more closely related to the strength of the masonry than would the prisms described above.

Recognizing the importance of the shape of the specimen on the recorded compressive strength of brick masonry, A.S.T.M. Committee C-15 appointed a Working Subcommittee on Tests for Brick Prism in 1937. This committee formulated a program of tests which were carried out at Columbia University under the direction of Professor W. J. Krefeld. The data obtained from these tests were reported by Professor Krefeld in a paper "Effect of Shape of Specimen on the Apparent Compressive Strength of Brick Masonry" which was appended to the 1938 report of A.S.T.M. Committee C-15. (A.S.T.M. Proceedings, Vol. 38, Part I.) The following is a summary of this paper:

"The brick used in the construction of all specimens were from one lot of soft mud, sand struck clay brick uniformly fired in a tunnel kiln, manufactured at New Britain, Conn. The physical properties of these bricks, as determined in accordance with standard procedure, are shown in Table 33.

The 'ratio of absorption' shown represents the weight of water absorbed after immersing one face of the dry brick in water to a depth of ⅛ in. for the time specified. This absorption is expressed in terms of grams per 30 sq. in. of surface immersed.

The mortar used for all specimens was a mixture of 0.85 portland cement: 0.15 lime putty: 3 sand by volume, representing a 1:3 mix with 15 per cent replacement of cement by lime putty. The corresponding mix by weight for the materials used is 1 portland cement: 0.152 lime putty: 3.6 sand. Specimens in the form of 2 by 2-in. cubes were

molded from batches used in the construction of each pier. Half of the specimens thus sampled were cured in water while the remainder were stored with the masonry under room temperature conditions and tested air dry.

**TABLE 33**

**PHYSICAL PROPERTIES OF CLAY BRICK**

| | Compressive Strength lb./sq. in. | Modulus of Rupture lb./sq. in. | Absorption % | | Ratio of Absorption C/B | Rate of Absorption, g. per 30 sq. in. | |
|---|---|---|---|---|---|---|---|
| | | | 24 hr. Cold | 5 hr. Boiling | | 1 min. | 3 min. |
| Maximum | 7600 | 1545 | 19.7 | 22.7 | 0.893 | 57.0 | 93.8 |
| Minimum | 3520 | 311 | 9.8 | 13.4 | 0.731 | 17.5 | 28.9 |
| Average | 5124 | 689 | 16.1 | 19.1 | 0.84 | 39.7 | 67.1 |

Two series of masonry piers were constructed with 8 by 16 in. and 12 by 16 in. cross-sections, respectively. A total of 30 piers included specimens with heights ranging from 8.5 to 97 in. (3 to 35 courses), as shown in Table 34. The variable dimensions provided specimens with $\frac{h}{d}$ ratios ranging from 1 to 12 approximately. All piers were laid with common bond including headers at every sixth course. The brick were laid dry with $\frac{3}{8}$ in. joints finished with a round jointer. The piers were constructed on 14 by 18 by $\frac{1}{2}$ in. flat steel plates and were capped with similar plates. These plates provided plane end bearing surfaces and also facilitated the handling. The specimens were stored at an average temperature of 70° F. and were tested air dry at 28 days.

The variation of compressive strength of brick masonry with $\frac{h}{d}$ ratio for both the 8 by 16 in. and 12 by 16 in. sections is represented by a single curve. These data indicate that for the sections tested, the "normal" compressive strength is obtained from specimens having an $\frac{h}{d}$ ratio of 6 or greater, when for all practical purposes the strength approaches a constant value. For an $\frac{h}{d}$ ratio less than 6, the apparent strength is influenced by a form factor. In the case of concrete cylinders, it is of interest to note that the strength is practically constant for $\frac{h}{d}$ ratios greater than 2, the usual 6 by 12 in. standard specimen. It would be expected that the strengths of both materials would decrease at some $\frac{h}{d}$ ratio larger than that investigated, when column action becomes effective.

Correction factors applicable to the test data here presented are shown in Table 35. These are presented as a guide pending further information showing the effect of other variables. The application of such correction factors would permit specimens of economical dimensions to be used for determination of compressive strength of brick masonry and reduction of results obtained from dissimilar specimens to a common basis of comparison.

## TABLE 34

## COMPRESSION TESTS ON BRICK MASONRY WITH VARIABLE HEIGHT-THICKNESS RATIOS

**Summary — Average (3 specimens)**

| Specimen | Nominal Section, Inches | Height Inches | Ratio Height to Thickness h/d | Comp. Strength lbs. per sq. in. | | Modulus of Elasticity lb./sq. in. | Mortar Strength lbs. per sq. in. | |
|---|---|---|---|---|---|---|---|---|
| | | | | First crack | Maximum Load | | Cured Wet | Cured Dry |
| Average.... | 8x16.5 | 8.72 | 1.09 | 2603 | 3424 | ..... | 2856 | 1523 |
| Average.... | 8x16.5 | 14.17 | 1.79 | 1757 | 2383 | ..... | 2928 | 1676 |
| Average.... | 8x16.5 | 25.29 | 3.18 | 1305 | 1925 | 840,800 | 2811 | 1689 |
| Average.... | 8x16.5 | 50.02 | 6.27 | 1212 | 1518 | 858,800 | 2628 | 1606 |
| Average.... | 8x16.5 | 97.3 | 12.17 | 1164 | 1402 | 869,300 | 2868 | 1630 |
| Average.... | 12x16.5 | 14.26 | 1.19 | 2107 | 2932 | ..... | 3034 | 1667 |
| Average.... | 12x16.5 | 25.26 | 2.12 | 1239 | 2213 | 1,020,300 | 2841 | 1594 |
| Average.... | 12x16.5 | 50.28 | 4.20 | 1280 | 1691 | 1,051,000 | 3047 | 1776 |
| Average.... | 12x16.5 | 97.0 | 8.09 | 1244 | 1488 | 881,700 | 2696 | 1847 |

## TABLE 35

## STRENGTH CORRECTION FACTORS FOR BRICK MASONRY PRISMS

| Ratio, Height to Thickness h/d | Strength Correction Factor | Ratio, Height to Thickness h/d | Strength Correction Factor |
|---|---|---|---|
| 1.1 | 0.45 | 4.5 | 0.93 |
| 1.5 | 0.59 | 5.0 | 0.96 |
| 2.0 | 0.67 | 5.5 | 0.98 |
| 2.5 | 0.75 | 6.0 | 1.00 |
| 3.0 | 0.80 | 8.0 | 1.03 |
| 3.5 | 0.85 | 10.0 | 1.06 |
| 4.0 | 0.89 | 12.0 | 1.09 |

**These data indicate that:**

1. The compressive strength of brick masonry is influenced by the $\frac{h}{d}$ ratio of the test specimen.

2. The variation of strength of 8 by 16 in. and 12 by 16 in. piers with $\frac{h}{d}$ ratio follows a common relationship.

3. The "normal" strength of masonry having these sections is recorded for an $\frac{h}{d}$ ratio of 6 or greater, and for smaller ratios the apparent strength is increasingly greater.

4. Suitable correction factors would permit the use of relatively short piers to be used for determination of the quality of the masonry. Although greater economy will result from the use of the smaller specimens, it is not advisable to adopt a specimen with $\frac{h}{d}$ ratio less than 3 or 4 due to the more prononuced influence of shape.

These conclusions are limited to the specimens tested. The influence of other factors such as brick and mortar, strength, bonding and other sectional dimensions, might well be included in a more extended program."

## 911. COMPRESSIVE STRENGTH

The results recorded in Table 30, indicate compressive stresses in flexure in brick masonry, ranging from 1740 to 2970 lbs. per sq. inch. H. Duff Williamson in Bulletin No. 46, Rensselaer Polytechnic Institute reports values of 2492 and 3486 lbs. per sq. in. From these results and others a compression value of 500 lbs. per sq. in. for the extreme fiber stress in compression seems justified for masonry built of materials which conform to minimum requirements and where the strength of the masonry is determined in advance of design this value might properly be increased.

## 912. BOND BETWEEN BRICK MASONRY AND STEEL

Tests for bond made by imbedding a reinforcing bar in a mortar joint of a small brick pier and then pulling the bar out, were conducted by Messrs. Parsons, Stang and McBurney of the National Bureau of Standards. The results of these tests are given in Table 31. Average ultimate bond strengths, at ages of one to two months, ranged from 870 to 950 lb./sq. in. Maximum bond stresses reported in Table 32 range from 1550 to 1842 lb. per sq. in. In the conclusions of R. P. No. 504, it is stated that differences in the kinds of brick and of curing conditions did not cause significant changes in bond strengths.

## 913. MODULUS OF ELASTICITY

Numerous tests made at college and commercial laboratories as well as the National Bureau of Standards (Table 28) indicate that the modulus of elasticity of brick masonry ranges from about 1,000,000 to more than 4,000,000 lb. per sq. in. The modulus of elasticity for concrete varies with the quality of the concrete and in a similar manner, the values for brick masonry vary with the quality of the brick, mortar, workmanship and method of bonding. Tests on reinforced brick masonry columns at the University of Wisconsin reported by Professor M. O. Withey at the 37th annual meeting of the American Society for Testing Materials gave the modulus of elasticity at one-fourth the ultimate load as ranging from 1,110,000 to 4,370,000 lb. per sq. in. There seems to be a relation between the modulus of elasticity and the ultimate compressive strength of brick masonry, and while this relation is not constant for all conditions, for design purposes it may be taken as 1000 times the ultimate compressive strength of the brickwork, assumed as the strength of a test specimen having a height to thickness ratio of 6 or over.

The ratio of the modulus of elasticity of steel to the modulus of elasticity of brick masonry ($E_s/E_b$) is denoted by the letter "n". The modulus of elasticity of all grades of steel is practically constant and commonly taken as 30,000,000. Tests of reinforced brick masonry indicate that the probable values for modulus of elasticity to be used in design range from 1,200,000 to 2,500,000, therefore, the recommended values for "n" range from 25 for masonry with an ultimate compressive strength of 1200 lb. per sq. in. to 12 for masonry with an ultimate compressive strength of 3000 lb. per sq. in.

## 914. SHEARING STRESSES IN BEAMS

In the aforementioned Research Paper No. 504, "Shear Tests on Reinforced Brick Masonry Beams", values for shearing stress in eighteen beams are reported (Table 27). Brick from two localities were used, both made from surface clay and formed by the stiff-mud, end-cut process. Twelve of the beams were constructed in the normal horizontal position, while six were built on end, somewhat like a column. Tests on these latter six beams indicate that this type of beam construction is not as good as those built in the normal horizontal position. The shearing stresses developed in the beams built in the horizontal position ranged from 80 to 171 lb./sq. in.

Tests on brick masonry beams made at the University of Wisconsin (Table 30) gave shearing stresses ranging from 91 to 294 pounds per sq. in.

## 915. SUMMARY OF PERFORMANCE PROPERTIES OF REINFORCED BRICK MASONRY

(a) Results of numerous experiments conducted on reinforced brick masonry members verify the original assumptions that reinforced brick masonry is similar in performance to reinforced concrete. The following conclusions are given in Professor Withey's report on Reinforced Brick Masonry Beams.

"1. The results of these tests show that it is possible to develop a high degree of flexural strength in reinforced brick beams.

2. With proper design of stirrup and longitudinal reinforcement, coefficients of resistance, $M/bd^2$, in excess of 500 lbs. per sq. in. and maximum shear stresses, v, in excess of 200 lbs. per sq. in. were obtained with all three varieties of brick. The maximum values of $M/bd^2$ and v were 840 and 294 lbs. per sq. in., respectively.

3. Where web reinforcement is required and no longitudinal rods are bent up, a stirrup spacing approximately one-half the effective depth of the beam is satisfactory.

4. The tests indicate that the formulas used in the calculation of fiber, shear, and bond stresses and deflections for reinforced concrete beams can, with proper constants, be used in like calculations for reinforced brick beams.

5. In order to secure good strengths particular attention must be given to see that all joints are completely filled.

6. In designing the arrangement of coursing, it is preferable to eliminate headers from heavily compressed portions of beams.

7. The 1 lime: 3 cement: 12 sand mortar used in these experiments gave very satisfactory results for this class of work.

8. With the mortar used in these tests, shear strengths were developed without stirrups of 93, 104 and 146 lb. per sq. in. in the beams of Chicago, Waupaca and Streator brick, respectively.

9. Brick of the Chicago type when dry should be sprinkled prior to laying to secure best results.

10. A knowledge of the moisture content desirable in a given brick and of the bond of the mortar to it is essential to proper design of reinforced brick beams.

11. Although conditions in these tests were not ideal for efficient bricklaying, the rate of operation indicated that with proper design of reinforcement excellent speeds can be attained in laying reinforced brick beams."

(b) Professor J. R. Shanks in his report after testing reinforced brick masonry beams at Ohio State University notes the following indications:

"The straight-line principle for fiber deformations does apply.

For proper bonding of the brickwork, the tensile aid to the steel from the brickwork is high, as high or higher than for reinforced concrete.

High moduli of elasticity can be had from strong hard brick and properly made, placed, and cured mortar.

There is little plastic flow in reinforced brickwork, much smaller than for concrete.

The stress known as diagonal tension in reinforced concrete is not present in reinforced brickwork for brick and mortar of quality such as used here. The corresponding stresses are shear and tension along and across the mortar joint, but this tension is not greater than the unit shear times the cosine of 45 degrees.

There are no good reasons apparent why reinforced brickwork cannot be designed and constructed in such a manner as to meet all structural requirements of safety, durability, and economy, as well as of beauty."

(c) Professor John W. Whittemore and Paul S. Dear made "A Comparison of the Performance Characteristics of Reinforced Brick Masonry Slabs and Reinforced Concrete Slabs" at Virginia Polytechnic Institute. Their summary and conclusions are given.

## SUMMARY

"1. The deflection characteristics of the reinforced brick masonry slabs compare very favorably with those of the reinforced concrete slabs. In practically every case the allowable deflection of 1/360 of the span was not reached until after the visible initial sign of failure of the slabs occurred.

2. The reinforced brick masonry slabs exhibit a slightly greater capacity for immediate deflection recovery throughout the alternate loading and unloading employed during the tests. During the early states of loading, however, the recovery characteristics are almost identical.

3. From the viewpoint of induced tensile stresses and strains in the reinforcing steel, the performance characteristics of the two types of construction are distinctly similar, in very close agreement, and well within safe limits, even considerably above design loads and bending moments.

4. From the viewpoint of induced compressive stresses and strains in the masonry (brickwork and concrete included) at the top surfaces of the slabs, the performance characteristics of both types of construction are also strikingly similar, in very close agreement, and well within safe limits, even considerably above design loads and bending moments.

5. Investigation of the behavior of the neutral axes of the slabs again shows distinctly similar performance characteristics.

6. From the beginning of application of live load until well after design live loads are reached the live load-deflection relationship follows an approximate straight line law.

7. During the same interval of loading, the relationship between bending moment and steel strain follows a straight line law; the relationship between bending moment and masonry strain (brickwork and concrete included) follows a straight line law; and the neutral axes of the slabs remain practically constant in their respective locations.

8. The experimentally determined tensile and compressive stresses for both types of construction, while varying rather widely from the stresses determined from theoretical formulas, are in close agreement with each other and also decidedly on the side of safety. The explanation of this variation in actual and theoretical stresses is explained fully in V. P. I. Engineering Experiment Station Bulletin No. 9, and applies equally to both types of construction.

9. The experimentally determined shearing and bond stresses are in very close agreement for the two types of construction. They are also in very close agreement with the corresponding theoretical shearing and bond stresses.

10. In both types of construction, all of the above mentioned experimentally determined stresses remain below their respective allowable limits until well past design loads and bending moments.

11. In both types of construction, loads greater than twice the design live loads are attained before the graphically indicated initial sign of failure is reached.

12. In both types of construction, visible signs of failure are not evident until loads several times greater than design loads are reached.

13. The maximum loads supported by the slabs demonstrate that both types of construction possess very ample safety factors when reinforced concrete theory is used as basis of design.

## CONCLUSIONS

1. Over ranges of loading well past design loads, the slabs of both types of construction perform like homogeneous beams in that all relations of load and moment to deflection and stress are linear.

2. The slabs of both types of construction possess ample stiffness even well above design loads.

3. From the viewpoint of induced stresses, both types of construction exhibit very ample safety factors when reinforced concrete theory is used as the basis of design.

4. The various performance characteristics of both types of construction are strikingly similar and in very close agreement throughout the entire range of loading.

5. The formulas of reinforced concrete design are adaptable and applicable to the design of reinforced brick masonry slabs.

6. The conclusion which formed the basis of this investigation — "Reinforced brick masonry slabs perform in a very similar manner to reinforced concrete slabs and are, therefore, theoretically and experimentally practicable," is confirmed fully and completely."

## 916. ALLOWABLE UNIT WORKING STRESSES IN R-B-M

While it is evident that the ultimate strength of reinforced brick masonry and therefore the maximum permissible working stresses depend upon the quality of the materials and workmanship entering into the construction, from the data available, it is possible to assume stress values which can safely be used in designing reinforced brick masonry construction using materials and workmanship which meet certain minimum requirements.

The following allowable Unit Working Stresses and regulations relating to reinforced brick masonry are included in the Uniform Building Code of the Pacific Coast Building Officials Conference as part of Section 2411.

| Type of Stress | 1C-½L-4½S Mortar | 1C-4½S Cement Grout |
|---|---|---|
| | Pounds per sq. in. | |
| Compression (Extreme fibre stress in bending) | 400 | 500 |
| Direct Compression on Piers | 300 | 400 |
| Shear (no web reinforcement) | 25 | 30 |
| Shear (with web reinforcement, taking entire shear) | 50 | 60 |
| Bond: Deformed bars, Vertical bars | 60 | 60 |
| Horizontal bars | 80 | 80 |
| Modulus of Elasticity E | 1,200,000 | 1,500,000 |
| Modulus of Rigidity G (Modulus of Elasticity in shear) | 480,000 | 600,000 |

The above working stresses are based upon minimum requirements for materials and may be safely used in the design of reinforced brick masonry structures where the properties of the materials entering into the construction are unknown except that they will at least conform to the minimum requirements of standard A.S.T.M. specifications.

In many buildings other considerations than strength will influence wall thickness and increases in working stresses will not materially affect the design. In such cases the above stresses may be taken as limiting values and the designer need not determine in advance the ultimate strengths of the materials to be used provided they conform to minimum A.S.T.M. requirements.

In some structure, however, where strength is a principal factor in the design, substantial economies may be effected through the use of higher working stresses. For such structures it is recommended that the designer determine in advance of the design the ultimate compressive strength of the masonry built of the same materials, brick, mortar, grout and with the same workmanship that will be used in the proposed construction. Working stresses may then be assumed based upon the known strength of the masonry. See Section 303, for probable strength of masonry.

The following allowable unit stresses, based upon the ultimate compressive strength of masonry are believed to be safe.

## TABLE 36

## ALLOWABLE UNIT WORKING STRESSES

### Reinforced Brick Masonry

| Type of Stress | Symbol | Approximate % of $f'_b$ | Allowable Unit Stresses in Brick Masonry, when Ultimate Strength ($f'_b$) as determined and controlled by Test Prisms amounts to: | | | | | |
|---|---|---|---|---|---|---|---|---|
| Ultimate Compressive Strength | $f'_b$ | | 1200 | 1500 | 1800 | 2100 | 2400 | 3000 |
| **Flexure:** | | | | | | | | |
| Extreme fiber stress in compression | $f_b$ | 33⅓ | 400 | 500 | 600 | 700 | 800 | 1000 |
| Extreme fiber stress in compression adjacent to supports of continuous or fixed beams or of rigid frames | $f_b$ | 40 | 500 | 600 | 700 | 800 | 950 | 1200 |
| **Bearing: (1)** | | | | | | | | |
| On full area | $f_b$ | 25 | 300 | 375 | 450 | 525 | 600 | 750 |
| On one-third area | $f_b$ | 33⅓ | 400 | 500 | 600 | 700 | 800 | 1000 |
| Pedestals (See Chapter 11) | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| **Shear:** | | | | | | | | |
| Beams with no web reinforcement and without special anchorage of longitudinal steel | v | 2 | 25 | 30 | 35 | 40 | 50 | 60 |
| Beam with no web reinforcement, but with special anchorage of longitudinal steel | v | 3 | 35 | 45 | 55 | 60 | 75 | 90 |
| Beams with properly designed web reinforcement but without special anchorage of longitudinal steel | v | 4 | 50 | 60 | 70 | 80 | 100 | 120 |
| Beams with properly designed web reinforcement and with special anchorage of longitudinal steel | v | 6 | 70 | 90 | 110 | 120 | 150 | 180 |
| Footings where longitudinal bars have no special anchorage | v | 2 | 25 | 30 | 35 | 40 | 50 | 60 |
| Footings where longitudinal bars have special anchorage | v | 3 | 35 | 45 | 55 | 60 | 75 | 90 |
| **Bond: (2)** | | | | | | | | |
| In beams and slabs and one-way footings: | | | | | | | | |
| Plain bars or structural shapes | u | 5 | 60 | 75 | 90 | 105 | 120 | 150 |
| Deformed bars | u | 6 | 75 | 90 | 105 | 125 | 140 | 175 |
| In two-way footings: | | | | | | | | |
| Plain bars or structural shapes | u | 3 | 35 | 45 | 50 | 60 | 70 | 90 |
| Deformed bars | u | 3¾ | 45 | 55 | 65 | 80 | 90 | 110 |
| Modular Ratio ($E_s/E_b$) | n | ..... | 25 | 20 | 18 | 15 | 15 | 12 |

(1) The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

(2) Where special anchorage is provided, one and one-half times these values in bond may be used.

### 917. ULTIMATE COMPRESSIVE STRENGTH

Brick masonry prisms or test specimens for determining the ultimate compressive strength ($f'_b$) of the masonry should be constructed of the same materials and with the same workmanship and, in so far as possible, the same bonding arrangement that will be used in the finished structure. It is recommended that all test specimens have height to thickness (h/d) ratios of 3 or over; however where facilities for testing or other conditions make this requirement impracticable, specimens of h/d ratios less than 3 may be used provided suitable correction factors are applied to their apparent strengths. All ultimate compressive strengths ($f'_b$) should be expressed as the equivalent strength of a specimen having an h/d ratio of 6 or more. This value may be obtained from the ultimate strength of a specimen of h/d ratio less than 6 by multiplying the strength of the specimen by the correction factor for the particular h/d ratio indicated in Table 35, Section 910. The following forms of test specimen are recommended:

For beams and slabs, in which the brick are laid as stretchers, test specimens 8″ x 8″ x 25″ constructed as outlined in Section 910 so that the brick will be in end compression.

For walls, test specimens 8″ x 16″ x 25″ high constructed in the same manner and with the same type of bonding that will be used in the structure so that the brick (as a rule) will be in flatwise compression.

For columns, 8″ x 8″ x 24″ high constructed in so far as possible with the same bonding arrangement that will be used in the column so that the brick (as a rule) will be in flatwise compression.

Brick masonry test specimens should be cured in air and tested at the age of 28 days.

The method of capping should be the same as that prescribed in the current standard Specification for Testing Brick, A.S.T.M. C67.

### 918. ALLOWABLE UNIT STRESSES IN REINFORCEMENT

(a) The unit stresses ($f_s$) in reinforcing steel should not exceed the following:

**In Tension**

| | | |
|---|---|---|
| Reinforcing bars | $f_s$ = | 20,000 lbs. per sq. in. |
| Web reinforcement | $f_s$ = | 20,000 lbs. per sq. in. |
| Structural steel shapes | $f_s$ = | 18,000 lbs. per sq. in. |
| Welded wire fabric or other steel reinforcement not exceeding ¼″ in diameter, fifty per cent of the minimum yield point as established by A.S.T.M. Standards for that particular grade of steel, but not to exceed | $f_s$ = | 30,000 lbs. per sq. in. |

**In Compression**

| | | |
|---|---|---|
| Structural steel section in combination columns | $f_s$ = | 16,000 lbs. per sq. in. |
| Reinforcing bars | $f_s$ = | 18,000 lbs. per sq. in. |

NOTE: Unit stresses in excess of 16,000 lbs. per sq. in. in the reinforcing for storage bins, silos or water tanks, are not recommended.

## CONSTRUCTION METHODS AND DETAILS

### 919. GENERAL

Keeping in mind the fundamental concept that reinforced brick masonry functions structurally in the same general manner as does reinforced concrete and that the same

design formulas may be applied, it follows that many of the construction methods and precautions commonly observed for reinforced concrete will apply to the construction of reinforced brick masonry. In this section, however, attention will be given more particularly to the detailed technique of brick laying and bar placing and to such other miscellaneous details as are especially applicable to the construction of reinforced brick masonry.

## 920. LAYING OUT BRICKWORK

Practically all R-B-M will be dimensioned on the basis of standard size brick — (2¼″ high, 3¾″ wide and 8″ long) with the majority of the mortar joints ½″ thick; there may be some ⅜″ and ⅝″ joints and very few ¾″. It is apparent, therefore, that the brickwork in R-B-M is practically the same as in plain brick masonry. The brick mason lays out his brickwork in the same manner; checking the brick for slight variation in size and arranging them so that any variations in length are compensating, rather than accumulative.

Mortar joints containing reinforcing rods should be wide enough to allow at least $\frac{1}{16}$″ of mortar between the brick and the rods. Special shaped brick, such as the Groutlock brick with beveled edges, provide extra space for reinforcing rods without increasing the joint thickness. With normal, standard brick, it is customary to make the mortar joint ⅛″ thicker than the diameter of the reinforcing rod.

## 921. BRICK STORAGE

The brick upon delivery to the job, should be stacked or piled on suitable planking and not dumped on the ground or into a puddle of mud or water. During freezing weather, the brick stock pile should be covered with a watertight tarpaulin or other suitable weathertight covering. Care should be taken to prevent the accumulation in the brick of any snow or ice. Also the brick stock pile should be kept away from driveways or, if this is impractical, it should be adequately protected by tight boards or other suitable materials so as to keep the brick pile from all splashing of mud, oil and other deleterious material.

## 922. WETTING OF BRICK

As a general rule, the lower absorption brick require little or no wetting before laying, whereas the higher absorption brick may demand an appreciable degree of wetting. The rate of absorption is the important factor and not the total absorption obtained by complete immersion of a brick in cold water for 24 or 48 hours, or the 5-hour boiling test. In other words, the amount of water that a brick absorbs during the first few minutes, is far more important in its relation to mortar strength and to the bond strength between brick and mortar than its total absorption.

The average brick will absorb about three-fourths of its 24-hour total (complete immersion in cold water at room temperature) in from 5 to 20 minutes. When it can be arranged, it is preferable that the brick which require wetting be soaked thoroughly the night before, or approximately four to eight hours before using. It is best to allow the brick in the pile to lose their excess moisture naturally. Therefore, the brick pile should be protected against drying winds and sunshine as well as heavy rains.

In case the brick are not given the previous soaking and allowed to stand a sufficient period (approximately four hours), the brick should be wetted but not soaked, before using. The amount of such last-minute wetting of the brick pile necessary to satisfy the brick absorption will, of course, vary with the particular type of brick used. For brick, having an average total 24-hour cold water absorption of about 15%, sprinkling with an ordinary garden hose for five minutes per 1000 brick, with periodic sprinkling thereafter, should approximate the desired result.

It is believed that equal success will be had with other types of brick by increasing the sprinkling for those with higher absorption properties and decreasing it for the less absorptive brick.

For the more impervious brick, say for example those having a cold-water absorption less than about 8% at the end of 24 hours, it is probable that no particular advantage is gained by wetting the brick much beyond the sprinkling necessary to remove kiln dust or other foreign substances.

### 923. CLEAN BRICK

Individual brick in reinforced brick masonry may be considered the coarse aggregate of R-B-M, occupying a similar position to that of stone or gravel in reinforced concrete, it follows, and for the same reason, that the brick surfaces should be free from oil, grease and other deleterious foreign materials. Good bond between brick and mortar cannot possibly be obtained when dirty brick are used. Second hand bricks, as a rule, are not suitable for reinforced brick masonry.

### 924. REINFORCING BARS

Obviously all reinforcing steel, before being incorporated in the brickwork, should be free from all loose mill or rust scale and from mud, oil or grease.

In all other respects, the usual rules pertaining to cutting, banding and marking of bars, as established for reinforced concrete construction, apply with equal force to reinforced brick masonry construction.

The typical hooked ends commonly used for reinforcing bars may, in some cases, be difficult to bed properly. In their place, square end bends of proper length have been successfully used. Such bends are much easier to mortar or grout into place. Tests made on such bars have shown the square bend to be sufficiently effective to resist end slip of bars at the increased unit bond stresses usually permitted for full hooked end bars.

The placing of loose vertical stirrups in reinforced brick masonry is not difficult. Two conditions in design, however, present themselves. If the reinforced brick masonry beam is to be covered with plaster or other facing material, then the "U" type of stirrups, commonly used for reinforced concrete can be introduced easily into the brickwork, but, as will be obvious from a little study, the brick cannot be laid in the usual running bond and the several vertical mortar joints will be in a continuous straight line extending from top to bottom of the beam at intervals of about 4 to 8 inches instead of being staggered in the usual way.

To overcome this difficulty, the use of a "Z" type stirrup was first suggested by Professor M. O. Withey of the University of Wisconsin. The "Z" stirrup is placed in the vertical mortar joint between the wythes of brick and under the longitudinal reinforcing. In tests and in actual construction, the "Z" type of stirrup has worked out satisfactorily and is recommended for general use, since the stirrup spacing can be varied at will and the stirrups can be doubled up if necessary to provide greater steel area. Spot welding the outstanding leg of the "Z" stirrups to the longitudinal steel will reduce labor on the job and may prove to be an advantage.

### 925. PLACING REINFORCING STEEL

The location of reinforcement in the joints of brick masonry has the advantage of greater control in the placement of steel in accord with the design. All reinforcing steel should be firmly imbedded in the mortar or grout.

Horizontal rods are laid in a smooth bed of mortar, tamped or pressed firmly into place and then covered by additional mortar spread to complete the full joint thickness before laying the next course of brick. In R-B-M beams containing stirrups, the "Z" bars are spaced according to plan with the lower horizontal leg immediately under the longitudinal horizontal reinforcing rods. If the stirrups are long, they may be wired or

tied to a board or rod at the top, which serves as a template holding the stirrups firmly in place. This template is removed when the brickwork is carried up to within a course or two of the top horizontal leg of the stirrups. If the stirrups are short, the template or support at the top may be omitted because the first few courses of brick above the base leg of the stirrups will provide sufficient support to maintain the stirrups in the vertical position.

In R-B-M walls, reinforcing rods are usually placed in both horizontal and vertical directions. The horizontal rods are placed in the designated joints by imbedding in mortar similar to beam construction. The vertical rods are placed and properly spaced by templates which are firmly supported. The templates may be boards with holes or slots at proper intervals or steel rods to which the vertical rods are wired.

It is not necessary to wire the vertical and horizontal reinforcing rods as in reinforced concrete. The horizontal rods need not be placed until the designated number of brick courses are built and the mortar bed spread as previously described.

R-B-M footings and foundations offer no new problem. Vertical rods or dowels usually extend above the brickwork. R-B-M walls built upon R-B-M or reinforced concrete footings and foundations will usually require the vertical reinforcing rods to be lapped and securely wired to the vertical dowels or rods extending from the masonry below.

In R-B-M walls it is not always possible nor convenient to use rods long enough to extend the full length or height. Therefore, convenient lengths are used and the stresses transmitted from one rod to another by sufficient overlapping. The amount of overlap is determined by the calculated stress in the rod and the allowable unit bond. Under most conditions, a lap of 40 diameters will be adequate.

## 926. FORMS

In general, the forms for reinforced brick masonry are the same as would be used for similar work in reinforced concrete. It has been found, however, that somewhat less bracing is required for reinforced brick masonry due to less superimposed weight, less tendency for the brickwork to become displaced and the greater relative rigidity of the masonry during laying, particularly in the case of beams. It has been found desirable in practice to cut all beam and slab forms about ¼ inch short at each end where such forms abut a wall or other member and to fill in such space either with a lime putty or with a very weak lime mortar. When the forms are removed this filling crumbles and the forms will not be found locked against the adjacent masonry due to the swelling of the form lumber.

The decking for slab forms can be made of any material possessing sufficient strength to carry the superimposed loads. Its thickness will, of course, be governed by the spacing of the supporting joists or caps. With a suitable spacing of joists, ⅜-inch ply wood, oiled to prevent absorption of moisture, will be found to give excellent results. The laying of so-called water-proofed building papers over square edged boards, to prevent the escape of mortar, has not worked out satisfactorily as there is a tendency for such paper to stick to the fresh brickwork and to discolor it. The sides of beams, columns and walls do not require forms.

In the mortar and grout-filled masonry, the horizontal beds of mortar are trowelled flat, the same as for stiff mortar construction. The outside edge of the vertical joints is feathered or buttered with mortar, then the grout is poured in, completely filling the interior joints in each course of brickwork. After the mortar has had time to harden partially, the exterior joints are tooled, preferably with a round jointer. The grout should be just fluid enough to flow freely around the reinforcement and to completely fill the joints. If the grout is too thin, it is slower in setting, and may produce hydrostatic pressures within the brickwork, especially when the brick have a low absorption and

the work is carried up very fast. Thin grout may be used with brick of fairly high absorption (15% or more), but with brick having absorptions of 8% or less, it is advisable to keep the grout as thick as possible. It is also recommended with grout-filled joints to arrange the work so that a new course of brickwork is not started until the grout in the last course has thickened considerably. In all brick, except those with low absorption, the grout will set fairly fast and cause no delay. With brick having low absorption, if the grout remains fluid for a long time, it is sometimes expedient to place wall ties in every 5th or 6th bed joint. These wall ties may be units of Z or U-bars, welded wire mesh, or a continuous wire or rod bent in a zigzag wave.

## 927. TYPICAL R-B-M CONSTRUCTION

In 1937, the Los Angeles, California Board of Education erected a reinforced brick masonry school building known as the Vermont Avenue School. The following description of the design and construction of this school prepared by Mr. Charles H. Fork, Consulting Structural Engineer, who collaborated with the Architects in its design, is presented as typical of the design and construction methods adaptable to reinforced brick masonry.

"The new Vermont Avenue School in Los Angeles, California, is an outstanding example of what can be done to make brick construction safe in an area subject to earthquake shocks.

The east and west wings of the schools are two stories high, connected in front with a one-story unit. The horizontal floor area is 38,700 sq. ft. Total cost of the building proper was approximately $180,000, or $4.62 per sq. ft. of floor area. The school cost less than a similar job of concrete. This is significant in view of the fact that face brick were used to face the 13-inch exterior walls and interior corridors.

Unusual features in this school are the use of shaped brick, and the grouting method for filling the interior joints.

The walls are distinguished from the usual type of reinforced brick masonry in that definite channels are provided for a grout core around the reinforcement. This is accomplished by the use of three different brick shapes: the standard unit for general filler purposes, the ¾ unit to provide grout space for the vertical reinforcement, and the ½ or narrow unit to provide grout space for horizontal reinforcement.

By the use of these units it is possible to place the reinforcement nearer the faces of the wall, and thus increase its effective depth. Greater assurance of securing a good bond between steel and masonry is obtained by the large grouting space. A very notable feature of the ¾ unit is its extreme flexibility for accommodating any spacing of vertical reinforcing bars. The bar spacing need no longer be in multiples of brick dimensions. If the unit laid in the usual manner does not clear the bar, the unit is simply reversed, thereby, moving the channel four inches forward. One unit provides a 4-inch channel, while two units placed adjacent to each other provide an 8-inch channel. In general, it is advisable to maintain a maximum bar spacing in order to provide sufficient working space for bricklaying. In this job a spacing of 2′2″ was used, wherever possible, for the vertical bars, and a spacing of 2′0″ for the horizontal bars. When it became necessary to increase the steel area either the size of the bar was made larger or the bars were grouped two in a channel.

The 'grouting' method was specified for the bricklaying to make certain that all joints would be filled and the reinforcing steel fully surrounded with grout. This method is now in general use locally and differs somewhat from ordinary bricklaying. Mortar of bedjoint consistency is used only for laying up the outer wythes. In laying up the 13-inch walls the outer wythes are kept ahead of the middle wythes, thus producing a continuous trough into which grout is poured. Brick for the middle course are then

shoved down into this grout and slightly vibrated, pushing the thin grout up on every side and solidly filling all crevices. It should be emphasized here that the grout, when poured, should be of a flowing consistency so it will run into all crevices and settle down by gravity against the vertical faces of the brick. This process, properly carried out, produces good adhesion on the various parts of the wall.

It may be reasoned that an excess moisture in the grout would be detrimental to its strength; however, since it is being poured within the limits of a porous form instead of a water-tight form, the excess moisture is removed by absorption in a few minutes. The immediate removal of the excess water (placing water) is an important asset, since it hastens a decrease in volume while the mortar is still green, permitting free adjustment. The actual shrinkage during the hardening process is then greatly reduced and internal stresses are minimized. The moisture taken in by the units surrounding the grout also provides a favorable curing condition. Numerous tests indicate that when grout is poured within the limits of a porous form, high strength is obtained.

All brick courses are stretcher courses with no headers used. Instead of connecting the wythes together by headers every fourth or sixth course the grouted method provides continuous connection between every course. In addition to the grouted connection between the wythes there is also a mechanical grout key provided wherever the channels occur for the reinforcing bars. It has been established by test that brick masonry strength is increased by the grouted method over that of the header method. The ratio of masonry strength to brick strength in grouted masonry is around .50, while for header masonry this ratio ranges from .25 to .33. This increase in strength may be directly attributed to the full connection between the wythes.

Each tier of reinforcing steel was completely set before the brickwork was started. The contractor's superintendent worked out a very simple, effective and economical method for bracing the reinforcement. The centering for the floors was erected in advance, to which was anchored a continuous piece for temporarily fastening the reinforcement. This device held the reinforcement rigidly and accurately in place and offered no interference to the bricklayer.

The only forms used were simple bottom forms for the lintels and spandrels. An interesting development was the use of the face brick course as a form. The brickwork was built up to a joint slightly above the floor line to serve as an outside form for the floor joists and slab. This method served a double purpose; in addition to serving as a form it provided a means of getting an intimate bond between the concrete and outside course of brick. Such methods establish unity of construction instead of numerous loosely connected parts. At the juncture of floor and walls every effort was made to provide continuity in both the walls and floors. The steel was continued through in both directions. Where brickwork was laid on concrete, shear keys were required. These keys were obtained by setting large brick units in the concrete immediately after it had been poured.

In stopping off work for a period of more than one hour, the specifications required that the top surface of the wall be entirely free of either mortar or grout, and the vertical joints kept down about an inch to form a key. This method produces a construction joint equivalent to any other joint or plane through the wall.

A cement mortar of one part cement, .25 parts lime putty and three parts sand, mixed by volume measurement, was used. The grout was one part cement, .15 parts lime putty and three parts sand. Mortar and grout containing slightly higher percentages of lime putty were also used in some walls.

The design of this building is based upon the lateral force requirements of the California State Division of Architecture; that is, the structure must not only support all gravity loads, but also must be able to resist 10% of these loads when applied laterally on the building in any direction. A definite path must be provided for transferring these

forces to the ground. The walls are suitably arranged to function in this manner. Longitudinally, paths are provided by the corridor walls and exterior side walls, and transversely, there are the end walls and cross walls. The design is generally satisfied when the lateral forces are applied; first, acting alternately in a transverse direction; and secondly, acting alternately in a longitudinal direction, on a building. When the force acts transversely, the longitudinal walls become vertical beams and are designed as such. The floor slabs provide the necessary reactions and are designed as horizontal girders to carry the accumulated forces to the transverse walls. These walls then act as vertical cantilevers for transferring the accumulated lateral forces to the ground. The same reasoning also applies when the lateral force acts in a longitudinal direction. It is seen that each wall is a continuous vertical beam for forces acting normal, and a vertical cantilever for forces acting parallel to the wall.

All the walls were used structurally to resist lateral forces and to support gravity loads; hence the resultant stresses were generally low. Only in a few cases did the stresses reach the following values:

| | |
|---|---|
| Direct compression | 175 lbs. per sq. in. |
| Combined stress (Direct and bending) | 292 lbs. per sq. in. |
| Shear (Reinforced) | 37 lbs. per sq. in. |
| Bond | 68 lbs. per sq. in. |

The exterior and interior walls are all bearing walls with an effective thickness of 13 inches for their entire height. The brickwork extends from the top of the spread footings to the top of the parapet wall. In the spandrel beams, the principal continuous steel is placed near the top and bottom, making the full depth available for beam action. The piers are treated in the same manner. This in effect makes the wall function as a continuous frame, enabling it to resist either tensile or compressive stresses.

In the reinforced concrete joist and slab construction for floors and roof, pans 30″ wide and 12″ deep were used. Joists were 30″ on center, with 3″ slab covering. As stated above, the slab is designed as a horizontal distributing girder for lateral forces and is suitably connected to the walls to resist its reactions.

One of the most important factors in producing sound brickwork is to find a mortar which will contain, in combination, the desirable qualities of strength, workability and water-retaining capacity. Lime putty added to a mortar, up to a point where it does not greatly affect its strength, is recommended. In the opinion of the author, the best combination of strength and workability can be obtained where approximately 65% to 70% of the cementitious material is Portland Cement, while the remaining 35% to 25% is of lime putty. Tests indicate that such a mortar has the necessary strength for reinforced brick masonry and at the same time possesses the equally important quality of good workability. A small amount of lime putty added to the grout also improves its flowability and helps to prevent segregation and keeps the mix in suspension for a longer time than in a straight mixture of sand, cement and water.

The rate of absorption of brick at time of being laid was given considerable study during the progress of the job. The common brick had a total average absorption of 10% to 16%, while the face brick had a total average absorption of about 10%. Brick were wetted the night before being laid. This had the effect of reducing the absorption rate and bringing the two kinds of brick roughly to the same absorption level at the time of laying. Moisture was added in sufficient quantities to bring the average absorption rate down to about 5% to 7%. The most favorable absorption rate can best be determined on the job by keeping the brick sufficiently dampened to build the wall up expeditiously and with no lateral 'floating.'

The quality of materials delivered to the job was maintained by frequent testing. In addition to the tests on individual materials, numerous supplementary tests were also made on brick prisms."

# CHAPTER 10

# BEAMS, LINTELS AND SLABS

## 1001. DISCUSSION OF DESIGN THEORY

Authorities agree that tests indicate the structural performance of reinforced brick masonry as being comparable with that of reinforced concrete and that the formulas used in the calculations of stresses and deflections in reinforced concrete beams can be used in similar calculations for reinforced brick beams. Also, as stated in the conclusions warranted after complete tests and comparisons of reinforced brick masonry and reinforced concrete slabs, both types of construction perform like homogeneous beams in that all relations of load and moment to deflection and stress are linear over ranges of loading well past design loads, and formulas for reinforced concrete design are adaptable and applicable to the design of reinforced brick masonry slabs.

Similar assumptions have been made in deriving the flexure formulas for reinforced brick masonry as are made in flexure calculations for reinforced concrete. Most text books on reinforced concrete design contain complete discussion of these assumptions and theory of design. In this volume, only an abridged derivation of the flexure formulas will be given.

## 1002. NOMENCLATURE

The following notations are used in the derivation of flexure formulas for working loads:

$A_s$ = effective cross sectional area of tension reinforcement in a beam or slab.

b = breadth of rectangular beam or slab.

d = depth from compression face of beam or slab to center of gravity of longitudinal tension reinforcement.

$E_s$ = modulus of elasticity of steel.

$E_b$ = modulus of elasticity of brickwork determined from tests made on compression prism specimens in which the brick are laid in the same position, relative to the direction of the applied stress, as in the beam or slab under consideration.

$n = E_s/E_b$ = ratio of modulus of elasticity of steel to that of brickwork.

$f_b$ = compressive unit stress in extreme fiber of brickwork.

$f_s$ = tensile unit stress in longitudinal reinforcement.

p = ratio of effective area of tension reinforcement, to the effective area of brickwork in a beam or slab $= A_s/bd$.

j = ratio of lever arm of resisting couple to depth d.

k = ratio of depth of neutral axis to depth d (from compressive face of beam or slab).

M = bending or resisting moment in general.

$M_s$ = resisting moment as determined by steel.

$M_b$ = resisting moment as determined by brickwork.

T = total tension in steel at any given vertical section of a beam.

C = total compression in brickwork at any given vertical section of a beam.

$e_s$ = unit deformation of steel due to $f_s$.

$e_b$ = unit deformation of brickwork due to $f_b$.

z = depth from compression surface of beam or slab to resultant of compressive stresses.

## 1003. FLEXURE FORMULAS FOR WORKING LOADS

The straight-line theory of stress distribution is universally accepted in the derivation of formulas for working stresses and safe loads for reinforced concrete beams and will be used in a similar manner for reinforced brick masonry beams. The unit stress in the steel is within the elastic limit. Plane sections before bending are assumed to remain plane sections after bending. The unit stress-deformations in the brick masonry at any given section of the beam are considered to vary as the ordinates to a straight line. These ordinates start with zero and increase directly with the distance from the neutral axis. Tension in the brick masonry is neglected.

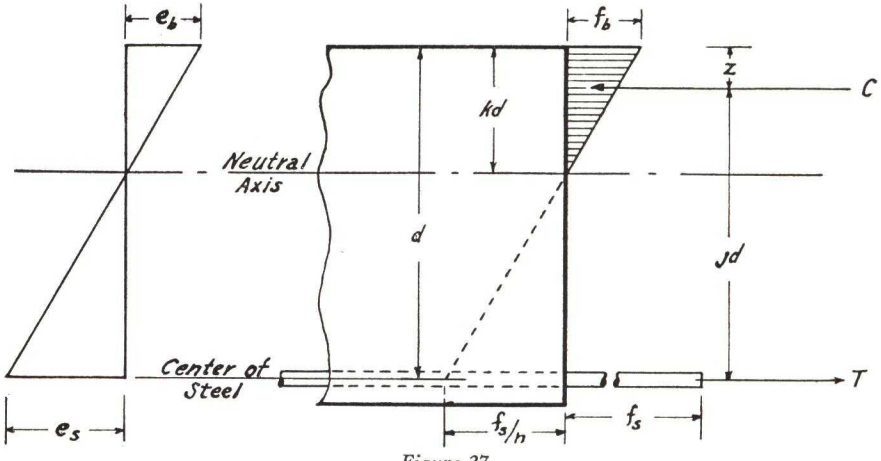



*Figure 27*

From the geometrical relations shown in Figure 27, it may be seen that:

$$e_s/e_b = (d\text{-}kd)/kd$$

Since $e_s = f_s/E_s$; and $e_b = f_b/E_b$ and $E_s/E_b = n$, we have:

$$\frac{f_s}{nf_b} = \frac{d\text{-}kd}{kd} = \frac{l\text{-}k}{k} \quad \ldots\ldots (1)$$

Under normal conditions of design, where the beam is level and all loads and reactions are vertical, the total compression (C) and the total tension (T) on a section are opposite and equal, therefore it follows that:

$$f_sA_s = \tfrac{1}{2} f_b\, kdb \quad \ldots\ldots (2)$$

Eliminating $f_s/f_b$ between equations (a) and (b) and introducing p, gives:

$$k^2 = 2pn(l\text{-}k) \quad \ldots\ldots (3)$$

Solving equation (c) results in:

$$k = \sqrt{2pn + (pn)^2} - pn \quad \ldots\ldots (4)$$

Equation (4) is basic and shows that the proportionate distance k which fixes the position of the neutral axis is the same for all beams having the same percentage of steel reinforcement and when built in the same manner of bricks and mortar having like physical characteristics.

Since the form of the compressive area shown in Figure 27 is that of a triangle, it follows that the distance, z, to the compressive resultant C is situated one-third of the distance down from the compressive face of the beam to the neutral axis, therefore, the arm of the resisting Couple C and T is:

$$j = l - \frac{k}{3} \quad \ldots\ldots (5)$$

As the value of k increases, the value of j decreases, but not in the same ratio, as may be seen by reference to Tables in the appendix, which show that k and j vary according to the values of p and n. It will be observed that j varies only slightly with p, and that for a value of n equal to 15, and with a variation in the value of p between the common design limits of 0.75 and 1.00%, the average value of j is very closely $\frac{7}{8}$. This fraction is commonly used by many designers for approximate designs and for checking.

If a beam is under-reinforced, its resistance to bending is limited by the allowable tensile stress in the steel reinforcement and its moment of resistance is:

$M_s = Tjd = A_s f_s jd = f_s pjbd^2$, from which is obtained:

$$f_s = \frac{M}{A_s jd} = \frac{M}{pjbd^2} \quad \ldots\ldots (6)$$

If a beam is over-reinforced, its resistance to bending is limited by the crushing strength of the brickwork and its moment of resistance is:

$M_b = Cjd = \frac{1}{2} f_b kdbjd = \frac{1}{2} f_b kjbd^2$, from which is obtained:

$$f_b = \frac{2M}{jkbd^2} = \frac{2pf_s}{k} \quad \ldots\ldots (7)$$

For approximate calculations and for checking, the average values of $j = \frac{7}{8}$ and $k = \frac{3}{8}$ may be used. Formulas (6) and (7) then become:

$M_s = \frac{7}{8} A_s f_s d$ ..... (6a), and

$M_b = 1/6\ f_b bd^2$ ..... (7a)

Which of the two resisting moments, $M_s$ or $M_b$, is the smaller must be determined in each specific case. A comparison of the values of $f_s p$ and $\frac{1}{2} f_b k$ is sufficient for such determination.

To design a beam or slab having equal resisting moment in tension and compression at working units stresses, the value of k is eliminated between equations (1) and (7) and equation is derived for p.

This value is:

$$p = \frac{\frac{1}{2}}{f_s/f_b\ (f_s/nf_b + 1)} \quad \ldots\ldots (8)$$

The value of p, for different values of $f_s$ and $f_b$ and for values of n = 10, 12, 15, 18, 20 and 25 is shown in Tables 78 to 82 in the appendix.

The cross section of a beam may easily be determined when M, $f_s$, $f_b$ and p are known or assumed. If the value of p is less than that given by equation (8), then the required cross section of a beam is:

$$bd^2 = \frac{M}{f_s pj} \quad \ldots\ldots (9)$$

If the value of p is greater than that given by equation (5), then the required cross section of a beam is:

$$bd^2 = \frac{M}{\frac{1}{2} f_b kj} \quad \ldots\ldots (10)$$

The values k and j can be obtained by using equations (4) and (5), or more readily from Tables No. 78 to 82 in the appendix.

It has been found convenient to substitute the letter K for the denominator in equations (9) and (10). The symbol K may properly be called the coefficient of resistance of the steel and brick work respectively and equations (9) and (10) may be re-written as:

$$bd^2 \tfrac{1}{2} M_s/K_s \ldots\ldots (9a) \ldots\ldots \text{and } bd^2 = M_b/K \ldots\ldots\ldots\ldots (10a)$$

Since for balanced design $M_s = M_b$, then $K_s = K_b$ and $K = \dfrac{M}{bd^2}$,

$$\text{or } bd^2 = M/K \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

For values of K, see Tables 78 to 82 in the appendix.

## 1004. SHEARING STRESSES

It has been found that shear is the principal factor in the failure of reinforced concrete beams by diagonal tension and should not be overlooked in reinforced brick masonry. In theory, bond stress is also a function of shear. The commonly accepted assumption is that shearing stress is the same at all points between the reinforcing steel and the neutral axis. The intensity of the horizontal shearing stress at any point is equal to that of the vertical shearing stress. Above the neutral axis, the shear decreases according to the parabolic law as in a homogeneous rectangular beam, becoming zero at the upper surface. The standard equation for the intensity of the shearing stress on any plane between the reinforcement and neutral axis is:

$$v = \frac{V}{bjd} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

where, v = the unit shearing stress
and V = the total shear
b, j and d are the same as given in Section 1002.
For approximate computations, the value of j may be taken as $\frac{7}{8}$. The equation then becomes:

$$v = \frac{8V}{7bd} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

## 1005. BOND STRESSES

The tension in the horizontal reinforcement near the lower surface of an R-B-M beam is maximum at the point of maximum moment. In a simple beam this is usually near the center, and decreases toward the supports. The difference in tension between any two points is transmitted from the steel to the brick masonry by the bond between the steel, mortar and brick. The equation for bond stress, in beams with horizontal reinforcement, is:

$$u = \frac{V}{\Sigma ojd} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

in which, u = bond stress per unit area.
$\Sigma o$ = sum of perimeters of all bars at a given section.

It is readily seen that the bond stress varies directly with the total shear (V) and inversely with the perimeters of the reinforcement bars ($\Sigma o$). Shear diagrams may be used to represent the variation of bond stress along a beam. Approximations may be made by using the average value of j as $\frac{7}{8}$.

If the reader is interested in a more complete discussion of the theory of shear and bond stress, he is referred to any standard text book on strength of materials.

## 1006. APPLICATION OF FORMULAS AND TABLES

### Illustrative Problem No. 1

Design an R-B-M slab for a span of 5′-6″ to support a uniform live load of 50 lbs. per sq. ft. Brick prism test specimens have shown an average ultimate compressive strength ($f'_b$) of 2200 lbs. per sq. in.; $f_s$ = 18,000 lbs./sq. in.

Solution: — The ultimate strength ($f'_b$ = 2200) of the prisms indicate that in the design, a safe working or allowable stress of 2200/3 = 733 lbs. per sq. in. may be used for the unit compressive strength of brick masonry. However, for convenience in using the tables, the next lower value listed in the tables ($f_b$ = 700) will be used. The use of this lower value will increase the safety factor.

The following values will therefore be used as a basis for design: (Section 929.)

$f_b$ = 700 lbs. per sq. in.
v = 40 lbs. per sq. in.
u = 105 lbs. per sq. in.
n = 15

and $f_s$ has been given as 18,000 lbs. per sq. in.

The weight of the brick masonry is assumed to be 125 lbs. per cu. ft.

Assume the thickness of slab as 2¼ inches (brick laid flat).

The dead load of the slab equals 125 x 2¼/12 = 23½ lbs. per sq. ft. Use 30 lbs. to allow for one-half inch of plaster on under side.

The total load (L.L. + D.L.) = 50 + 30 = 80 lbs. per sq. ft.

Designing the slab as a simple beam, one foot wide, (b = 12″) the maximum bending moment, $M = \frac{Wl^2}{8} \quad \frac{80 \times 5\frac{1}{2} \times 5\frac{1}{2} \times 12}{8} = 3{,}630$ in. lb.

Using formula (11) from Section 1003,

$bd^2 = M/K$
b = 12″
M = 3,630 in. lb.
K = 113.1 (See Table No. 81 in appendix)
Therefore,

$$d^2 = \frac{M}{bK} = \frac{3630}{12 \times 113.1} = 2.68$$

$$d = \sqrt{2.68} = 1.64''$$

with a thickness of brick equal to 2¼″, the distance to center of steel is 2.25−1.64 = 0.61″, almost ⅝″. A half inch is permissible when the bottom of the slab is protected with not less than ½″ of plaster.

Referring again to Table No. 81, a balanced design, when n = 15, $f_b$ = 700 and $f_s$ = 18,000, should have the following values:

p = 0.0072
k = 0.3684
j = 0.8772

The required area of steel, $A_s$ = pbd = 0.0072 x 12 x 1.64 = 0.1414 sq. in.

Formula No. 6 may be written in the form:

$$A_s = \frac{M}{f_s jd'}, \text{ and used as a check}$$

$$A_s = \frac{3630}{18{,}000 \times 0.877 \times 1.64} = 0.1404 \text{ sq. in.}$$

Referring to Table No. 76, it is found that ¼″ round bars in a half-inch mortar joint between bricks laid flat (3¾″ + ½″ = 4¼″ centers) has an area of steel per foot width of slab equal to 0.138 sq. in. This is slightly less than the required amount. From a practical standpoint, it would suffice. However, the actual effective depth, d, that is available, is 2¼″ — ½″ = 1¾″ or 1.75″, instead of the 1.64″ calculated as the minimum required. Using d as 1.75″ in the above formula,

$$A_s = \frac{3630}{18{,}000 \times 0.877 \times 1.75} = 0.1314 \text{ sq. in.}$$

Therefore, 0.138 sq. in. of steel per foot of slab width will be more than sufficient, provided the effective depth is made 1¾″.

The design will now be analyzed for the new values of $f_s$ and $f_b$.
with $A_s = 0.138$; b = 12″ and d = 1.75″

$$p = \frac{A_s}{bd} = \frac{0.138}{12 \times 1.75} = 0.66$$

Using formulas 4 and 5, when p = 0.66 and n = 15, it is noted that k = 0.356 and j = 0.881.

Substituting in Formula No. 6

$$f_s = \frac{M}{A_s jd} = \frac{3630}{0.138 \times 0.881 \times 1.75} = 17{,}080 \text{ lbs. per sq. in.}$$

The compressive stress in brick masonry is checked by substituting in Formula No. 7

$$f_b = \frac{2pf_s}{k} = \frac{2 \times 0.66 \times 17{,}080}{0.356} = 633 \text{ lbs. per sq. in.}$$

It is noted that the calculated unit stresses in the reinforcement and brickwork are less than the allowable unit stresses. The design is satisfactory from the standpoint of compressive stress in the brick masonry ($f_b$) and tensile stress in the steel ($f_s$), but will be checked for shear and bond stress.

The total shear (V), at the support, is

$$V = \frac{wl}{2} = \frac{80 \times 5\frac{1}{2}}{2} = 220 \text{ lbs.}$$

Unit shearing stress (v), using Equation 9

$$v = \frac{V}{bjd} = \frac{220}{12 \times 0.881 \times 1.75} = 10.85 \text{ lbs. per sq. in.}$$

The allowable unit shearing stress for the brick masonry in this problem is 40 lbs. per sq. in. No stirrups or bent-up bars will be necessary.

The unit bond stress (u) is determined by using Equation 11.

$$u = \frac{V}{\Sigma o jd}$$

The summation of perimeters ($\Sigma o$) is determined first with brick laid flat and ¼″ round bars in half-inch mortar joints, the centering of the bars is 3¾″ + ½″ = 4¼″. The total perimeters per 12″ width of slab is $\frac{0.785 \times 12}{4\frac{1}{4}} = 2.215″$. Another method is to multiply the perimeter of one bar by the area of steel ($A_s$) per foot width and divide by the area of one bar.

$$\Sigma o = \frac{0.785 \times 0.138}{0.049} = 2.21″$$

Substituting in Equation 11,

$$u = \frac{V}{\Sigma ojd} = \frac{220}{2.21 \times 0.881 \times 1.75} = 64.5 \text{ lbs. per sq. in.}$$

The allowable unit bond stress in this problem is 105 lbs. per sq. inch, and it will not be necessary to hook the bars or provide special anchorage.

Therefore, the design satisfies all conditions, and will consist of brick laid flat with ¼″ round bars in each ½″ mortar joint and placed 1¾″ below the top surface or ½″ above bottom surface of the brick. The under side of the slab is to be covered with plaster ½″ thick.

**Illustrative Problem No. 2**

A slab is made with brick on edge (3¾″ deep), in ½″ mortar joints. The ¼″ round reinforcement rods are placed in each joint and 3″ below the surface. The spacing of the bars is, therefore, 2¼″ + ½″ = 2¾″. Test prisms of the brickwork indicate an ultimate compressive strength ($f'_b$) of 1840 lbs. per sq. in. What is the safe uniform live load per sq. ft. of slab on a span of 8 feet? The allowable tensile stress in the reinforced steel ($f_s$) is 18,000 lbs. per sq. in.

Solution: — The slab will be analyzed as a simple beam 12″ wide. The resisting moment (M) will be determined for both the steel and brick masonry. The lesser of the two will be used to calculate the allowable load.

The ultimate strength ($f'_b$ = 1840) of the test prisms will permit the use of the following values:

$f_b$ = 600 lbs. per sq. in.
v = 35 lbs. per sq. in.
u = 90 lbs. per sq. in.
n = 18

Unless otherwise specified, the weight of the masonry is assumed to be 125 lbs. per cu. ft. The dead load per sq. ft. is, therefore, $125 \times \frac{3\frac{3}{4}}{12} = 39.1$ lbs. The reinforcement, ¼″ round bars on 2¾″ centers, makes the area of steel ($A_s$) per foot width,

$$A_s = \frac{0.049 \times 12}{2\frac{3}{4}} = 0.214 \text{ sq. in.}$$

$$p = \frac{A_s}{bd} = \frac{0.214}{12 \times 3} = .0060$$

and the summation of perimeters ($\Sigma o$) per foot width,

$$\Sigma o = \frac{0.785 \times 12}{2\frac{3}{4}} = 3.42 \text{ inches}$$

From Table No. 80, when n = 18, $f_b$ = 600 and $f_s$ = 18,000,
p = .0062; k = 0.375; j = 0.875 and k = 98.44

The maximum moment for the reinforcing steel,
$M_s = jA_sf_sd = 0.875 \times 0.214 \times 18{,}000 \times 3 = 10{,}120$ in. lb.

The maximum moment for the brick masonry
$M_b = \frac{1}{2}f_bkjbd^2 = \frac{1}{2}x600x0.375x0.875x12x3^2 = 10{,}640$ in. lb.

The smaller moment of 10,120 in. lb. (843.3 ft. lb.) will be used, and since the maximum moment in a simple beam with uniform load (w) is

$$M = \frac{wl^2}{8}, \text{ and solving for } w = \frac{8M}{l^2} = \frac{8 \times 843.3}{8 \times 8} = 105.3 \text{ lbs. per ft.}$$

The dead load has been calculated to be 39.1 lbs. per sq. ft. Therefore, the allowable live load will be 105.3 — 39.1 = 66.2 lbs. per sq. ft.

The slab will be checked to see whether or not this loading will cause excessive stresses in shear or bond.

The total shear:

$$V = \frac{wl}{2} = \frac{105.3 \times 8}{2} = 421 \text{ lbs.}$$

Unit shearing stress,

$$v = \frac{V}{bjd} = \frac{421}{12 \times 0.875 \times 3} = 13.36 \text{ lbs. per sq. in.}$$

The allowable shearing stress is 35 lbs. per sq. in.
Unit bond stress,

$$u = \frac{V}{\Sigma ojd} = \frac{421}{3.42 \times 0.875 \times 3} = 47 \text{ lbs. per sq. in.}$$

The allowable bond stress is 90 lbs. per sq. in. Therefore, the R-B-M slab described in this problem will safely support a uniform live load of 66 lbs. per sq. ft.

**Illustrative Problem No. 3**

Design an R-B-M beam with a span of 12 ft. to support the slab in Problem No. 2. Use $f_b = 600$, $f_s = 16{,}000$, $n = 18$.

Solution: — The slab in Problem No. 2 spans 8 feet; therefore, the beams are spaced on 8 ft. centers and support 8 ft. of slab per running foot. The total load from the slab is 105.3 x 8 = 842 lbs. per foot of beam. Assuming the dead load of the beam to be 150 lbs. per ft., the total dead and live load per foot of beam will be 842 + 150 = 992 lbs. (Use 1000 lbs. per lin. ft.) from Table No. 80, we find: p = 0.0076; k = 0.403; j = 0.866; K = 104.66

$$M = \frac{wl^2}{8} = \frac{1000 \times 12^2 \times 12}{8} = 216{,}000 \text{ in. lb.}$$

For a balanced design, Equation 11, gives

$$bd^2 = \frac{M}{K} = \frac{216{,}000}{104.66} = 2068$$

Assume b as 8 inches, then $d^2 = \frac{2068}{8} = 258.5$; $d = 16.1$

$A_s = pbd = 0.0076 \times 8 \times 16.1 = 0.978$ sq. in.

This will require five ½″ Ø = 0.982 sq. in.

Five bars on 1¼″ spacing, require 4 x 1¼″ = 5″ space for bars.

This will allow $\frac{8 - 5}{2} = 1\frac{1}{2}$″ cover at sides of beam.

With ½″ round bars, the mortar joint in which the steel is placed will be made ¾″ thick.

| | |
|---|---|
| 6 courses of brick @ 2¼″ | = 13½″ |
| 5 mortar joints @ ½″ | = 2½″ |
| ½ of first mortar joint | = ⅜″ |
| Effective depth (d) | = 16⅜″ |

The minimum effective depth required was 16.1″. Therefore, a depth of 16⅜″ is satisfactory. The total depth of beam is,

| | |
|---|---|
| Effective depth to center of steel | = 16⅜″ |
| ½″ of first mortar joint | = ⅜″ |
| First course of brick | = 2¼″ |
| Total overall depth | = 19″ |

$$\text{Weight of beam} = \frac{125 \times 8 \times 19}{144} = 132 \text{ lbs. per foot.}$$

Since the calculated weight is fairly close and less than the assumed weight, many designers would let the design ride with this difference considered as an additional factor of safety. However, a recheck will be made to see if the size of the beam can be reduced.

The total dead load plus live load is 842 + 132 = 974 lbs. per lin. ft.

$$M = \frac{wl^2}{8} = \frac{974 \times 12^2 \times 12}{8} = 210{,}500 \text{ in. lb.}$$

$$d^2 = \frac{M}{bk} = \frac{210500}{8 \times 104.66} = 251.2; \; d = 15.9''$$

$$A_s = pbd = 0.0076 \times 8 \times 15.9 = 0.966 \text{ sq. in.}$$

This area of steel requires five ½″ round bars as in original design. Using the same number of courses of brick, but decreasing the mortar joints from ½″ to ⅜″, will give an effective depth (d) = 15¾″. This is not sufficient. The original depth of 16⅜″ will be used.

The beam will now be checked for shearing stress.

$$V = \frac{wl}{2} = \frac{974 \times 12}{2} = 5844 \text{ lbs.}$$

$$v = \frac{V}{bjd} = \frac{5844}{8 \times 0.866 \times 16\frac{3}{8}''} = 51.5 \text{ lbs. per sq. in.}$$

| | |
|---|---|
| Allowable shear on brick was | = 35.0 lbs. per sq. in. |
| Shear to be taken by stirrups | 16.5 lbs. per sq. in. |

Try ¼″ round Z bar. The required spacing of stirrups at the supports,

$$s = \frac{A_v \times f_v}{vb} = \frac{0.05 \times 16{,}000}{16.5 \times 8} = 6.06 \text{ inches (use 6″)}$$

The first stirrup will be placed not more than ½ x 6″, or three inches from the beam support.

$$\text{No stirrups will be required at} \frac{12}{2} \times \frac{35}{51.5} = 4.08 \text{ ft. from the center of the span.}$$

Therefore, stirrups will be required up to 1.92 feet or 1′-11″ from the end of the beam.

If the beam is built with all stretcher courses, the stirrups may be spaced exactly as calculated. As in reinforced concrete design, a shear diagram may be used to determine stirrup spacing. However, this spacing can be approximated with a reasonable degree of accuracy. For instance, midway between the support and that point on the beam where no stirrups are required, the stirrup spacing is double that required at the support. In this problem, at a point 11½″ from the support, the maximum stirrup spacing is 2 x 6 = 12 inches. This gives the designer an idea of the rate of increase in stirrup spacing.

However, the maximum spacing should be approximately one-half the effective depth (d/2). In this problem, therefore, we shall locate the ¼″ Z-bar stirrups at the 3, 11 and 19 inches from the support.

If there is likely to be a header course in the beam, it is necessary to space the stirrups to fit in the mortar joints. With a brick 3¾″ wide and the usual half-inch joint, this spacing becomes some multiple of 4¼ inches. If the required spacing is less than 4¼″, a blind header is used at that location. In this problem, with a header course, the spacing could be made a maximum of 3″ from the support, then two spaces of 8½″, making the stirrup locations, 3″, 11½ and 20″ from the supports.

Designers will frequently find it convenient to place a quarter-inch rod horizontally in each interior vertical mortar joint in a beam immediately above or as part of the horizontal tension reinforcement. Thus a beam 8″ wide would have one ¼″ rod in the center of the reinforcement, while a 12″ beam would have two, and a 16″ beam would have three of such rods. These rods are then bent up near the supports to aid the stirrups in resisting diagonal tension.

More than one ¼″ horizontal rod may be placed in the mortar joint between the wythes of brick. The rods are then spaced from ¾″ to 1″ apart in a vertical plane. The upper rod is bent first, while the bend of the lowest rod is nearest the support,

The unit bond stress,

$$u = \frac{V}{\Sigma ojd} = \frac{5844}{7.86 \times 0.866 \times 16\frac{3}{8}} = 52.5 \text{ lbs. per sq. in.}$$

The allowable bond stress for plain bars is 90 lbs. per sq. in. No special anchorage will be required, but hooks will be made at the ends of a couple rods for additional safety.

### 1007. BEAM CHARTS

The charts for R-B-M Beams at the end of this chapter have been prepared for the more popular beam sections. Various design values for the allowable compressive stress ($f_b$) in brick masonry, allowable tensile stress ($f_s$) in reinforcement and ratio n, ($E_s/E_b$) have been used to obtain the data. The safe uniform loads are indicated for different simple spans, with the corresponding requirements in reinforcement. Suggestions are noted for the number and size of reinforcement bars. The abscissas represent the required area of steel and the designer may choose any combination of rods to provide the necessary area. Table No. 77 (Bar Sizes and Equivalent Areas) will be found convenient in choosing bar combinations for steel areas up to two square inches.

The transverse curved bands (AA, BB, etc.) are used as an aid in stirrup spacing. An example or two will illustrate the use of these charts.

#### Illustrative Problem No. 4

Design an R-B-M beam or lintel, eight inches wide to span five feet with a uniform load of 1900 pounds per linear foot, not including the weight of beam. Allowable unit stresses, $f_b = 500$, $f_s = 16{,}000$, $n = 20$. Weight of brick masonry is 117 lbs. per cu. ft.

Try Chart No. 7 for beam with corresponding values of $f_b$, $f_s$ and n. Weight of beam, $\frac{177 \times 8 \times 13\frac{3}{4}}{144} = 89.4$ lbs. (use 90 pounds per foot). Total load per foot is then 1900 + 90 = 1990 pounds per foot. Locate 1990 pounds on the ordinate or vertical scale of uniform loads, then the intersection of a horizontal line through 1990 and the line labeled "5-ft. span." This intersection occurs between the abscissas on the horizontal scale for 4-⅜″ round bars and 5-⅜″ round bars. Use 5-⅜″ round bars.

The intersection of 1990 on the five-foot span is also between the transverse bands representing stirrup spacings of 4¼″ and 2⅛″. It is possible to prorate the stirrup spacing, and in this case, our point of intersection lies approximately midway between

$2\frac{1}{8}''$ and $4\frac{1}{4}''$, so it will be safe to use three-inch spacing near the supports. The first stirrup ($\frac{1}{4}''$ Z-bar type as shown) will be placed $3'' \times \frac{1}{2}'' = 1\frac{1}{2}''$ from the support and then a three-inch space to the next. The distance from the support at which the stirrup spacing is changed will be determined in this manner:

We will call "X" the intersection along the five-foot span line with the 1990 ordinate, and similarly, "C","B" and "A", the intersection of stirrup lines CC, BB, and AA, respectively. The origin is "O". The distance OX graphically represents half the span (2'-6"). XC is the proportionate amount of OX which represents the distance from the support at which the stirrup spacing changes from 3" to $4\frac{1}{4}''$. In a like manner, CB represents the distance used for $4\frac{1}{4}''$ spacing, and BA the distance required for six-inch spacing. No stirrups are required within that distance to the center of span represented by OA.

In this particular problem, the stirrup spacing of three inches is carried $30'' \times XC/OX = 5\frac{3}{4}$ inches from the support. The $4\frac{1}{4}''$ spacing runs an additional distance of $30'' \times CB/OX = 3\frac{1}{4}$ inches, and six-inch spacing for $30'' \times AB/OX = 7\frac{1}{2}''$. No stirrups are required within $30'' \times OA/OX = 13\frac{1}{2}$ inches of the center, or $30'' - 13\frac{1}{2} = 16\frac{1}{2}''$ from the support. The stirrups in this example will be spaced $1\frac{1}{2}$ inches from the support, then three, four and five inches. Half of the six-inch spacing, or three inches, past the last stirrup brings us $16\frac{1}{2}''$ from the support or to a point $13\frac{1}{2}$ inches from the center, beyond which no stirrups are required.

If a header course interferes with stirrup spacing, such as given above, it is permissible to break the headers at stirrup locations, or the stirrups can be placed in the mortar joints. The spacing is then a width of brick and a mortar joint ($3\frac{3}{4}'' + \frac{1}{2}'' = 4\frac{1}{4}''$), or some multiple thereof. In the above example, two $\frac{1}{4}''$ Z-bars could be placed in the first joint away from the support. Then place one stirrup in each of the next three joints ($4\frac{1}{4}''$ spacing). If the first joint is more than two inches from the support, place a stirrup also in the first joint beyond the support.

### Alternate Designs

If it is possible to obtain a greater effective depth than 11 inches, such as $16\frac{1}{2}$ inches, try Chart No. 9 and check the following results, (Remember, weight of beam is now 125 pounds per foot). Reinforcement required: three $\frac{3}{8}''$ round bars; one stirrup placed $7'' \times \frac{1}{2}'' = 3\frac{1}{2}$ inches from support. No stirrups required beyond 9" from support. Place second stirrup in a mortar joint 4 to 6" from first stirrup.

In the shorter spans with maximum loads, it is probable that the bond stresses may exceed the allowable bond stress and therefore the reinforcement bars should have hooked ends. It is advisable, however, to hook the ends of at least two horizontal bars.

### Illustrative Problem No. 5

Design an R-B-M beam similar to Problem No. 3 in Section 1006. Given:

Live load = 842 lbs. per linear ft.
Span ($l$) = 12 ft.
$f_b$ = 600 lbs. per sq. in.
$f_s$ = 16,000 lbs. per sq. in.
n = 18

Weight of brick masonry = 125 lbs. per cu. ft.

Solution: — Try Chart No. 15. Weight of $8'' \times 19\frac{1}{4}''$ beam per foot (See Table No. 75) is 134 lbs. Total load is 842 + 134 = 976 lbs. per ft. Following the ordinate for a uniform load of 976 lbs. to its intersection with the 12-ft. span, the chart indicates that the required area of reinforcement steel is 0.92 sq. in. Referring to Table No. 77 (Bar Sizes and Equivalent Areas) it is noted that $19\text{-}\frac{1}{4}''\varnothing$ with an area of 0.933 sq. in., $5\text{-}\frac{1}{2}''\varnothing$ with an area of 0.982 sq. in., or $9\text{-}\frac{3}{8}''\varnothing$ with an area of 0.994 sq. in. will meet

the steel requirement. It will be impractical to attempt to place 19-¼″∅ or 9-⅜″∅ in a beam 8″ wide. The reinforcement will, therefore, consist of five ½″ round bars.

To determine the stirrup spacing, the intersection of the 12-ft. span with the ordinate for uniform load of 976 lbs. is marked. Designate this point as X. The distance, OX, from the origin to X, represents one-half the span, or 72″, and may be scaled on the chart in any convenient engineer's scale. Using a scale of 10 ft. to the inch, the distance OX scales 62, OA (to the AA line) scales 42 and OB scales 57. No stirrups will be necessary closer than $72 \times OA/OX = 72 \times \frac{42}{62} = 49$ inches from the center of span. The 8½″ stirrup spacing is required at $72 \times \frac{OB}{OX} = 72 \times \frac{57}{62} = 66$ inches from the center of span, or 6 inches from the support. The stirrup spacing required at the support may be interpolated for X between the lines BB (representing 8½″ spacing) and CC (representing 4¼″ spacing). In this instance, the required stirrup spacing at the support (represented by X) is approximately 1¾″ less than the 8½″ spacing, or 6¾ inches. Therefore, the first stirrup will be placed at 6¾ x ½ = 3⅜″ (use 3″) from the support. The next two stirrups will be spaced at 8½″ centers. The third stirrup will be 3 + 8½ + 8½ = 20 inches from the support, or 52 inches from the center of the span. This stirrup will easily take care of the diagonal tension in the remaining 3 inches, beyond which point no stirrups are necessary.

A comparison of the beam designed according to the chart and the one designed with flexure formulas or equations, will show a remarkable similarity. It is noted that the beam designed in Problem No. 3 (Section 1006) has an effective depth of 16⅜″, while the beam from Chart No. 15 has an effective depth (d) of 16½″. The horizontal reinforcement bars, as well as the stirrup spacing is identical.

### 1008. LINTELS

The designer is again reminded that the sections indicated on a chart are merely graphical representations of the beam size. The arrangement of brick may be varied to suit the particular conditions. It is not necessary to place headers in the base or any other course not in keeping with the general architectural treatment.

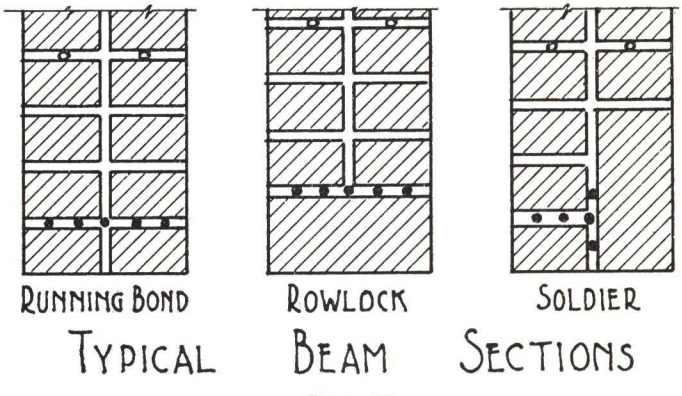



*Figure 28*

For lintels, many architects prefer a soldier course, rowlock or common running bond, as illustrated. Any one of these are readily obtained in R-B-M.

If a lintel supports the spandrel wall only, without any superimposed loads, most designers assume the area of wall supported by the lintel as triangular in form. This area is an isosceles triangle, where the equal angles at the sides are usually assumed as not less than 45°, nor more than 60°. The moment for the 45° triangle over the lintel is equivalent to the moment resulting from a uniform load in pounds per foot of one-third (⅓) the span times the weight of one square foot of wall area. In other words, the uniform load per foot to be used on the beam chart, when designing lintels supporting a spandrel wall, may be taken as $\frac{lw'}{3}$, where $l$ = span in feet, and w′ = weight of one square foot of wall area. When the triangular section is a 60° equilateral, the uniform load per foot to be used on the beam chart is $\sqrt{\frac{lw'}{3}}$ or 0.577lw′. An example will illustrate:

**Problem:** What is the equivalent uniform load per foot that will be used in designing a lintel having a span of six feet and supporting an 8-inch brick wall, with no superimposed loads?

**Solution:** Assume weight of brick masonry as 125 pounds per cubic foot and 45° isosceles triangular load on lintel.

An 8-inch wall weighs $\frac{125 \times 8}{12} = 83.3$ pounds per square foot. (Use 85 pounds per square foot.)

The equivalent uniform load would be $\frac{lw'}{3} = \frac{6 \times 85}{3} = 170$ pounds per foot.

In a similar manner, a 9-foot lintel in an 8-inch wall would be designed for a uniform load of $\frac{85 \times 9}{3}$ or 255 pounds per foot, and a 12-foot lintel for $\frac{85 \times 12}{3}$ or 340 pounds per foot.

The designer is reminded of the fact that on the basis of the 45° triangular loading, a wall having a height of less than one-third the span is more economically designed for the actual load per foot of lintel.

Using the chart for 8-inch by 13¾-inch R-B-M beams and lintels, the equivalent uniform load of 170 pounds per foot on a 6-foot span will require less than four ¼-inch round bars for reinforcement. In fact, two ¼-inch bars will suffice. On the same chart, it is found that the load of 255 pounds per foot on a 9-foot span requires four ¼-inch round bars. A 12-foot span is not shown on this chart. Inasmuch as the load of 340 pounds per foot is based on a height of wall equal to one-third the span, or four feet, the beam or lintel will have an effective depth greater than 11 inches. The same is true for the six and 9-foot lintels, but the problems have been worked out to illustrate the use of the chart.

Using the chart for 8-inch by 19¼-inch R-B-M beams to design the 6, 9 and 12-foot lintels, with their respective loads of 170, 255 and 340 pounds per foot, it is found that the 6-foot lintel requires only one ¼-inch round bar; the 9-foot lintel, three ¼-inch bars, while the 12-foot lintel will need three ⅜-inch round bars.

In determining the number and sizes of bars required, it is permissible to interpolate, or use bars whose sectional area is equivalent to the required area indicated on the chart. For instance, an 8-inch by 19¼-inch lintel on the 6-foot span requires a steel area of 0.035 square inches. One ¼-inch round bar is ample with 0.049 square inches sectional area. It would be preferable, however, to use welded steel mesh composed of four No. 12 wires (BWG gauge) or two No. 9 wires. All steel should be free from rust,

dirt and grease and should extend over the jamb for a distance equal approximately to the effective depth as shown on the chart.

The charts are for simple or freely supported beams. Lintels are actually restrained to some extent. The positive moment for a restrained beam would be less than that for a simple beam. It is safer, however, to design lintels for positive moment as in simple beams.

The negative moment of a restrained lintel should not be overlooked. While calculations may be made to determine the amount of negative moment, as a general thing, one-half the area of steel used for positive moment will suffice for negative moment. It is recommended that the bars for negative moment be placed in a mortar joint at a distance above the bottom of the lintel at least equal to the effective depth used for positive moment.

For instance, the 12-foot lintel designed according to the chart for 8-inch by 19¼-inch R-B-M beams, requires three ⅜-inch round bars (or six ¼-inch bars) for positive moment. The effective depth is shown as 1 foot-4½ inches. The length of the bars should be 12 feet plus twice 1 foot-4½ inches or approximately 15 feet.

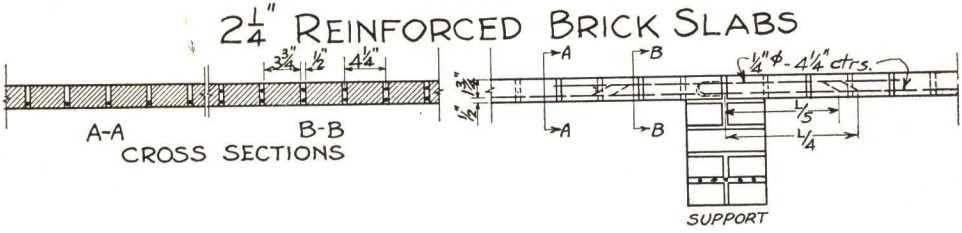
Three or four ¼-inch round bars will be used for negative moment, and should extend from the quarter point to 1 foot-6 inches or 2 feet past the jamb. It is a good plan to extend a couple of these ¼-inch bars entirely around the building, making 18-inch laps where necessary. These bars serve as tie-beams and tend to distribute any concentrated loads.

## 1009. SLAB DESIGN TABLES

Problems No. 1 and 2 in Section 1006 illustrate the design of two R-B-M slabs. Tables 37, 38 and 39 give the safe uniform live loads per sq. ft. on slabs having thicknesses of 2¼", 3¾" and 6½", and computed for several values of $f_b$ and $f_s$. The weight of reinforced brick masonry has been assumed to be 125 lbs. per cu. ft. in computing the dead load of the slab. If the under side of the slab is plastered, the weight of such plaster per sq. ft. will have to be deducted from the allowable live load.

Column A contains safe loads for the slab designed as a simple span, using the equation for maximum moment, as $M = \frac{wl^2}{8}$. Column B is for the end span of two or more continuous spans, using the equation for maximum moment, $M = \frac{wl^2}{10}$. The loads listed in Column C are computed for the interior of continuous spans, using $M = \frac{wl^2}{12}$. It is recommended that approximately half of the reinforcement be bent up and carried near the top of the slab over end supports.

The negative moment in the slabs at the face of intermediate supports of continuous spans is approximately equal to $\frac{wl^2}{12}$. It is evident that an equal amount of reinforcement will be necessary near the top of slabs over intermediate supports as is used at the bottom of the slab near the center of the span. One method of placing the steel is for alternate bars to be bent up and carried near the top of slab over supports, while the intermediate bars run straight through. In the adjoining slab, the process is reversed, so that alternate bars run straight through under the bent up bars of the first described slab, and intermediate bars are bent up and carried over the bars which come straight through from the other slab. The bars are bent up at a distance from the support ranging from one-fifth to one-fourth of the span, and carried a similar distance beyond the support.




## TABLE NO. 37

### 2¼" REINFORCED BRICK SLABS

### SAFE UNIFORM LIVE LOADS IN LB./SQ. FT.

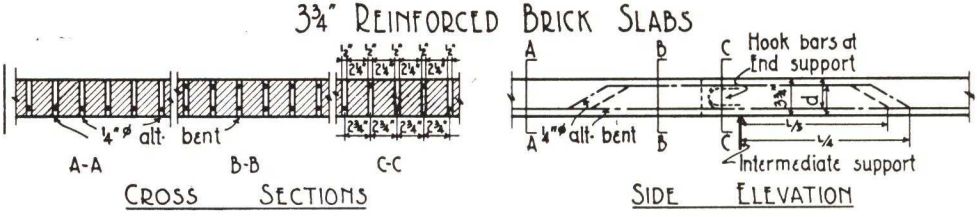
**Column A for Simple Spans; Column B for End Spans; Column C for Continuous Spans**

| Span | n = 20 $f_b$ = 500 $f_s$ = 16000 | | | n = 18 $f_b$ = 600 $f_s$ = 16000 | | | n = 18 $f_b$ = 600 $f_s$ = 18000 | | | n = 15 $f_b$ = 700 $f_s$ = 18000 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | A | B | C | A | B | C | A | B | C | A | B | C |
| 2′-0″ | 375 | 400 | 425 | 450 | 480 | 500 | 500 | 550 | 600 | 600 | 650 | 700 |
| 3′-0″ | 200 | 240 | 280 | 220 | 260 | 300 | 240 | 280 | 325 | 250 | 320 | 375 |
| 4′-0″ | 100 | 140 | 175 | 115 | 150 | 185 | 125 | 165 | 200 | 130 | 170 | 210 |
| 5′-0″ | 58 | 75 | 95 | 65 | 87 | 110 | 73 | 95 | 120 | 78 | 100 | 125 |
| 5′-6″ | 44 | 61 | 75 | 50 | 65 | 85 | 56 | 75 | 93 | 60 | 80 | 100 |
| 6′-0″ | 33 | 44 | 60 | 38 | 54 | 69 | 43 | 60 | 75 | 47 | 64 | 80 |

Loads above the double lines require hooked ends on reinforcement bars, or some other form of special anchorage.

Illustrative Problems No. 1 and 2 of Section 1006 may be checked by these tables. Problem No. 1 specifies a uniform load of 50 lbs. per sq. ft. on a span of 5′-6″, with $f_b$ = 700 and $f_s$ = 18,000. Under Column A of the extreme right section of the table for 2¼" brick slabs, it is noted that the safe uniform load on a span of 5′-6″, is 60 lbs. per sq. ft. The weight of a half-inch of plaster would be 5 or 6 lbs. per sq. ft., leaving a safe load of 54 or 55 lbs. The additional few pounds safe load may be accounted for when one considers that the slabs are analyzed and the safe loads computed on the basis of the allowable unit stress in both reinforced brick masonry and steel. In the problem, the slab, as designed, actually had a small surplus of steel area ($A_s$) over that required.

Solving Problem No. 2 by beams of the table for 3¾" R-B-M Slabs, it is found that for $f_b$ = 600 and $f_s$ = 18,000, the safe uniform load for a span of 8 feet is 65 lbs. per sq. ft. Our previous solution by flexure formulas gave the safe load as 66 lbs. per sq. ft.




## TABLE NO. 38

### 3¾" REINFORCED BRICK SLABS

### SAFE UNIFORM LIVE LOADS IN LB./SQ. FT.

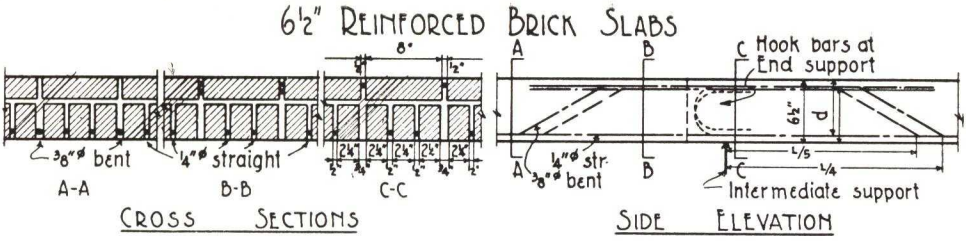
**Column A for Simple Spans; Column B for End Spans; Column C for Continuous Spans**

| Span in Feet | $f_b=500$; n = 20; $f_s=16{,}000$; d = 3" | | | $f_b=600$; n = 18; $f_s=18{,}000$; d = 3" | | | $f_b=700$; n = 15; $f_s=20{,}000$; d = 3" | | | $f_b=600$; n = 18; $f_s=18{,}000$; d = 2¾" | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | A | B | C | A | B | C | A | B | C | A | B | C |
| 3 | 620 | 790 | 950 | 700 | 890 | 1080 | ..... | ..... | ..... | 620 | 780 | 940 |
| 4 | 330 | 425 | 500 | 380 | 480 | 590 | 430 | 550 | 670 | 330 | 420 | 490 |
| 5 | 200 | 260 | 320 | 230 | 295 | 365 | 260 | 340 | 410 | 200 | 250 | 300 |
| 6 | 125 | 165 | 200 | 145 | 195 | 240 | 170 | 220 | 275 | 125 | 160 | 200 |
| 7 | 80 | 100 | 140 | 95 | 130 | 150 | 115 | 150 | 180 | 80 | 100 | 130 |
| 8 | 50 | 75 | 100 | 65 | 90 | 115 | 79 | 105 | 135 | 50 | 75 | 100 |
| 9 | ..... | ..... | ..... | 43 | 65 | 85 | 54 | 75 | 100 | ..... | ..... | ..... |
| 10 | ..... | ..... | ..... | 28 | 45 | 60 | 36 | 54 | 73 | ..... | ..... | ..... |

Loads above the double lines require hooked ends on reinforcement bars.

Slabs made with more than one course of brick, permit the use of various patterns in the top course. The basket weave and common bond illustrated are the two patterns commonly used. They are easily laid and the continuous joints permit the placing of steel near the top when needed for negative moment over the supports or in cantilever spans. Slabs, simply supported, which do not need reinforcement for negative moments, may have a top course in a variety of brick patterns, with disregard for any continuity of the joints.

With the common bond pattern in the top course, or by inverting the section so that the top course is composed of bricks on edge, it is possible to use the 6½" slab in cantilever construction. Slabs made with brick on edge (3¾" deep) are also used in cantilever construction for canopies over porches, balconies and entrances, or as projecting bands used in modernistic architecture.




## TABLE NO. 39

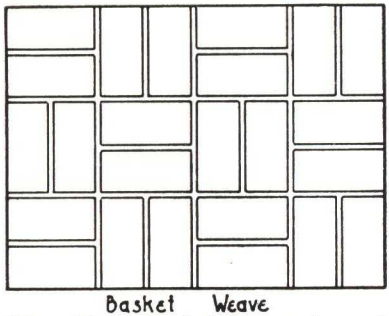
### 6½" REINFORCED BRICK SLABS

### SAFE UNIFORM LIVE LOAD IN LBS./SQ. FT.

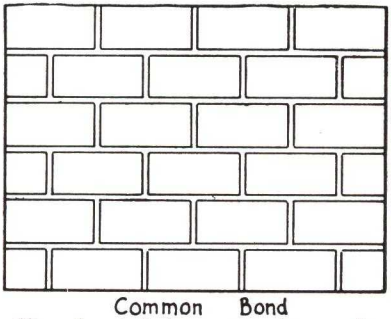
**Column A for Simple Spans; Column B for End Spans; Column C for Continuous Spans**

| Span in Feet | $f_b$ = 500; n = 20; $f_s$ = 16,000; d = 5¼" | | | $f_b$ = 600; n = 18; $f_s$ = 18,000; d = 5¼" | | | $f_b$ = 700; n = 15; $f_s$ = 20,000; d = 5¼" | | |
|---|---|---|---|---|---|---|---|---|---|
| | A | B | C | A | B | C | A | B | C |
| 5 | 650 | 830 | 1000 | 745 | 950 | 1150 | 875 | 1050 | 1300 |
| 6 | 425 | 550 | 675 | 490 | 630 | 775 | 565 | 720 | 875 |
| 7 | 300 | 350 | 475 | 345 | 440 | 550 | 400 | 500 | 625 |
| 8 | 210 | 275 | 300 | 245 | 320 | 375 | 280 | 375 | 450 |
| 9 | 150 | 195 | 250 | 190 | 250 | 310 | 210 | 280 | 350 |
| 10 | 110 | 150 | 200 | 145 | 180 | 230 | 160 | 215 | 270 |
| 11 | 80 | 115 | 150 | 100 | 140 | 180 | 120 | 160 | 200 |
| 12 | 55 | 85 | 115 | 75 | 110 | 145 | 90 | 125 | 160 |
| 13 | 39 | 65 | 90 | 50 | 80 | 110 | 65 | 100 | 135 |
| 14 | 24 | 45 | 70 | 35 | 60 | 85 | 45 | 75 | 105 |

Loads above the double lines require hooked ends on reinforcement bars.



Basket Weave
Other 8" square designs may be used



Common Bond
Other bond designs may be used

## 1010. R-B-M SLAB WITH METAL LATH

Another type of R-B-M slab has been developed by J. H. Hansen and Odd Albert (The American Swedish Engineer, October 1935) which consists of one course of brick laid on ribbed metal lath. The metal lath, serving as both form and positive reinforcement, is clipped to the supports and then covered with a bed of stiff mortar, spread evenly to a thickness which will cover the rib at least one-eighth of an inch. The brick are then laid flat or on edge, with ½″ joints. Small rods (¼″) may be dropped in each joint to augment the positive reinforcement, and tamped sufficiently to imbed each rod in the mortar, before the joints are filled with grout.

Tests were made in the summer of 1935 at New York University on this type of a slab, with brick laid flat. Three span lengths were tested, 5′, 5′6″, and 6′. See Engineering News-Record, August 22, 1935. The actual deflection at maximum loading was safely under the allowable deflection (1/360 span) specified in most building codes. At working loads, this deflection would be only a fraction of the allowable deflection. The equivalent uniform load at failure was 651 lbs. per sq. ft. for the 6′ span, 775 lbs. per sq. ft. for the 5′6″ span, and 739 lbs. per sq. ft. for the 5′ span. It is obvious that under the usual building code requirements, this type of construction can be used with a large factor of safety as a floor slab or roof deck in practically all types of buildings.

As in other types of slabs, when this particular type is built as a continuous span, it is necessary to bend up the reinforcement rods and carry them near the top of the slab over the supports in order to resist negative moment. The construction is simple and can be efficiently executed. The floor surface may be ground, polished and waxed, or covered with any type of flooring material. The underside of the metal lath is given a coat of plaster.

## 1011. GENERAL DISCUSSION OF R-B-M BEAMS AND SLABS

As previously stated, the beam sections on the R-B-M Beam and Lintel charts are intended to present graphical illustrations of the method of constructing such beams and the placing of reinforcement. The dimensions for the beam width (b) and effective depth of steel (d) are typical for the loads and spans plotted, but in all other respects, the details of design may be varied to meet individual conditions. Reinforced brick masonry, when properly constructed of suitable brick, mortar and grout, becomes a homogeneous mass in which the usual requirements for masonry or header bonding may be disregarded. This freedom in the use of headers simplifies the placing of reinforcement and permits the laying of brick in any position.

Beam widths are established by the thickness of the brick wythes and the mortar joints between these wythes. Any desired width of beam may be obtained by the proper combination of wythes and thickness of joints. For the purpose of illustration, standard brick dimensions and ½″ mortar or grout thickness result in a wide range of widths. Since header bond courses are not imperative, these widths may be increased to meet exact requirements by simply increasing the thickness of the vertical joint.

The safe loads for beams and lintels of widths other than those plotted on the charts may be easily determined from the charts. The loads and steel areas are proportionate to the loads and steel areas plotted, with the effective depths (d) being the same. For example, the safe load on a beam 10¾″ (10.75″) wide will be $\frac{10.75}{12.25}$ of the indicated load on an 8″ beam of the same span and effective depth, and the required area of steel will be $\frac{10.75}{12.25}$ of the indicated steel area in the 12.25″ beam.

The effective depths given on the charts were determined on the basis of standard brick sizes and ½″ joint thickness. Brick and joint sizes may vary, but in selecting a suitable beam from the chart, it must be borne in mind that the given effective depth

must be maintained as the minimum. The nominal overall depth of the beam may vary, depending upon the position in which the bottom or soffit course of brick is laid, flat or on edge. The effective depth is measured to the center of gravity of the group of reinforcing bars, which should be accurately determined for beams of limited depth, such as floor beams, but is not so important in wall beams, where the wall above acts as part of the beam and serves to increase the effective depth.

The charts and their sections present a suggested structural design of R-B-M beams and lintels. The bottom mortar bed is shown 1″ in thickness, which is ample to accommodate reinforcing bars of any size up to ¾ inch. In unexposed work or certain types of structure, this heavy bed may not be objectionable, but this thickness is not necessarily compulsory. The minimum thickness of mortar beds and joints in which longitudinal reinforcement is embedded need be only ⅛″ more than the largest bar diameter, to provide sufficient mortar or grout coverage or embedment on all sides of the steel.

Uniform thickness of exposed mortar beds may be maintained without difficulty. The exposed brick may be clipped at the back and backup brick laid with a thick bed for the steel, making up the difference by reducing the bed thicknesses above. The second course of the beam may be formed with split brick (soap) at the exposed faces, forming a large space in which the reinforcement may be properly spaced and grouted. Special brick shapes may be used to form a similar space for the reinforcement.

The architectural design of bond or pattern for exposed beam surfaces is unrestricted. Longitudinal reinforcement must, of course, be placed in straight beds, but such beds are usually available, and stirrups are placed in the vertical interior joints. Headers may be of either whole or half brick. Beam soffits may be of stretcher, header, rowlock or soldier courses, with square, rounded or molded corners. The complete filling of all interior joints and spaces with mortar and grout provides ample bond to keep all units in place. This has been demonstrated in tests in which soffit brick remained solidly in place after failure of the beam.

In R-B-M floor slabs, the architectural pattern is governed to a greater extent by the structural design. Straight continuous joints are necessary for the placing of the reinforcement in slabs one brick thick. Depending upon the structural design, the pattern may consist of straight courses of brick on edge or flat with the end joints lined or staggered as desired, or square patterns such as basket weave.

## 1012. CONSTRUCTION PROCEDURE

Designers and contractors will undoubtedly develop considerable efficiency and many improvements in the technique of reinforced brick masonry construction. Good workmanship is essential. All mortar beds should be spread evenly and all joints should be completely filled. Exterior joints should be tooled, preferably concave with a round jointer. The following procedure has been used satisfactorily in building R-B-M beams and slabs:

### Beams

(a) Lay one course of brick with full mortar joints on top of the soffit form.

(b) Immediately thereafter spread a smooth bed of mortar on this first brick course and then lay the longitudinal reinforcing rods on the mortar bed in correct position and press them down evenly into the mortar. If the diameter of the rod is only ⅛″ less than the joint thickness, the rod may be pressed down until it rests on the upper bed face of the first course of bricks.

(c) If stirrups are required, slip the outstanding leg of the "Z" stirrups under the longitudinal reinforcing bars, taking care that the vertical leg of the stirrup is on the exact center line of the vertical mortar joint into which it is to be built. If "U" type

stirrups are used, they should be laid down on top of the first brick course, at correct spacing, the longitudinal bars superimposed thereon, and then bedded in correct position with stiff mortar.

(d) After the longitudinal bars are in correct position, spread additional mortar over them, if necessary, fill all voids and depressions caused by the placing of the reinforcing steel and make a smooth bed for the next course.

(e) Lay the second and all succeeding courses of brick required to make up the dimensioned depth of the beam in the usual way, making sure that there is mortar or grout around the stirrups. If all mortar is used in the joints, the construction is similar to that employed in any other good brick masonry.

(f) Grout-filled joints are made by laying each course of brick in an even bed of mortar, buttering the head joints. The inside vertical joints of each course are then filled with grout, and the process repeated.

**Slabs**

(a) In building slabs, it has been found both practical and expedient to use a grout instead of mortar. The brick for a single thickness slab are started along one edge of a slab and laid directly on the forms without bedding of any kind. For obtaining the desired width of mortar joints between brick courses, successful use has been made of a gas pipe, an electrical conduit or a smooth reinforcing rod of the proper size. After a row of brick is placed, the spacer (rod, pipe or conduit) is removed to the outside, ready for the next row. Some builders prefer to slush a small quantity of mortar down between the brick immediately after removing the spacer, or to dispense with spacers and spread some mortar on the side of each brick at the bottom edge before setting in place. The mortar is then tamped well in order to fill the bottom of the joint to a uniform height, depending upon the required location of the rods. This mortar tends to keep the brick in proper position, seals the bottom of the joint and serves as a bed for the reinforcement rods, which are then dropped in place. If mortar is not placed in the bottom between the brick courses, wire hangers of the "U" type may be placed at proper intervals and the reinforcing rods in turn dropped into the open space between the brick, supported in the correct position by these hangers. After all reinforcement is in position, it is important to seal all exterior vertical joints by pointing with mortar before pouring the grout.

(b) If the brick to be used for a slab are fairly absorbent, it is well to wet them as recommended for all good masonry. It is also advantageous to paint the top surface of the brick with a thin coating of paraffin oil or with a thin sizing of clay. This will prevent the adherence of the mortar to the finished surface and will result in a finished floor surface which will show the natural color and texture of the individual bricks. After the mortar has set, the finished slab surface is washed off with clear water. If the brick surface is to be ground and polished, it is not necessary to take this precaution of painting with oil or clay.

(c) The grout used for the mortar joints is poured from a bucket or other suitable vessel directly into the open joints and should be just fluid enough to run freely around the steel and fill the joint. Grout is subject to considerable shrinkage and may tend to congeal and not flow completely into all the voids. Therefore, after a joint has been once filled, it will be necessary to run a trowel or pencil rod along the joint to aid the flow of grout and probably go back a second time to fill in the space caused by such shrinkage.

(d) If a slab of two courses of brick is to be built, proceed as outlined above. Then spread a fairly stiff bed of mortar for the middle or horizontal joint, using wood or pipe screeds of specified thickness for leveling off. Into this horizontal mortar joint

may be placed all transverse or temperature rods and also the conduits for electric wiring. The top course of brick is then laid in the same manner as described above and reinforced as required by the plans.

(c) A very beautiful floor finish may be obtained by filling the uppermost ½ inch or ¾ inch of the mortar joints with a terrazzo floor mixture and then grinding, filling and waxing. This terrazzo mixture may be had in any color. A number of such floors have already been built at a very reasonable cost and are gaining in popularity. A basket weave bond pattern for the brick slab lends itself to easy placement of reinforcement, particularly where two-way reinforcement is required. The usual running bond pattern in a one-course slab can be used for one-way reinforcement only.

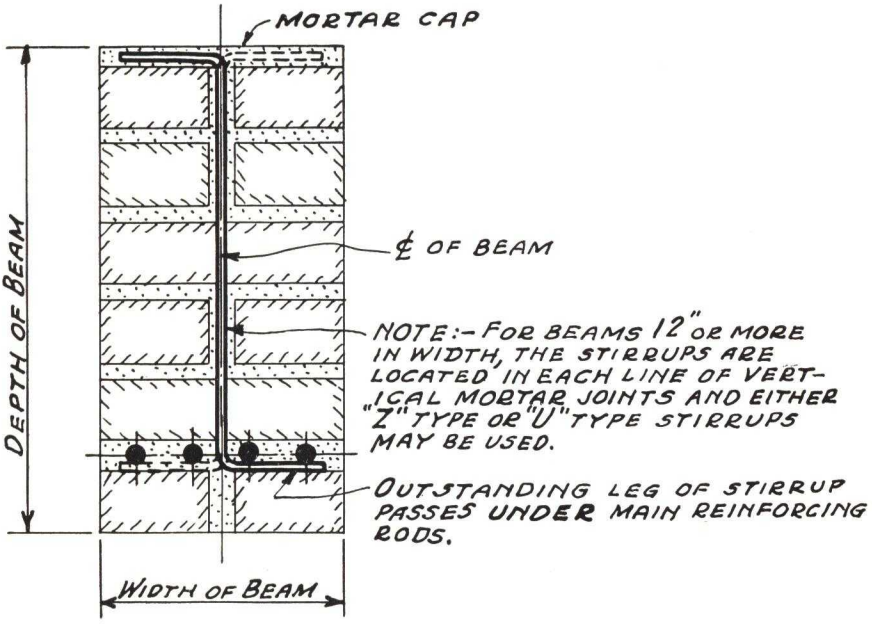



*Figure 29*

*Showing reinforcement in R-B-M beam*

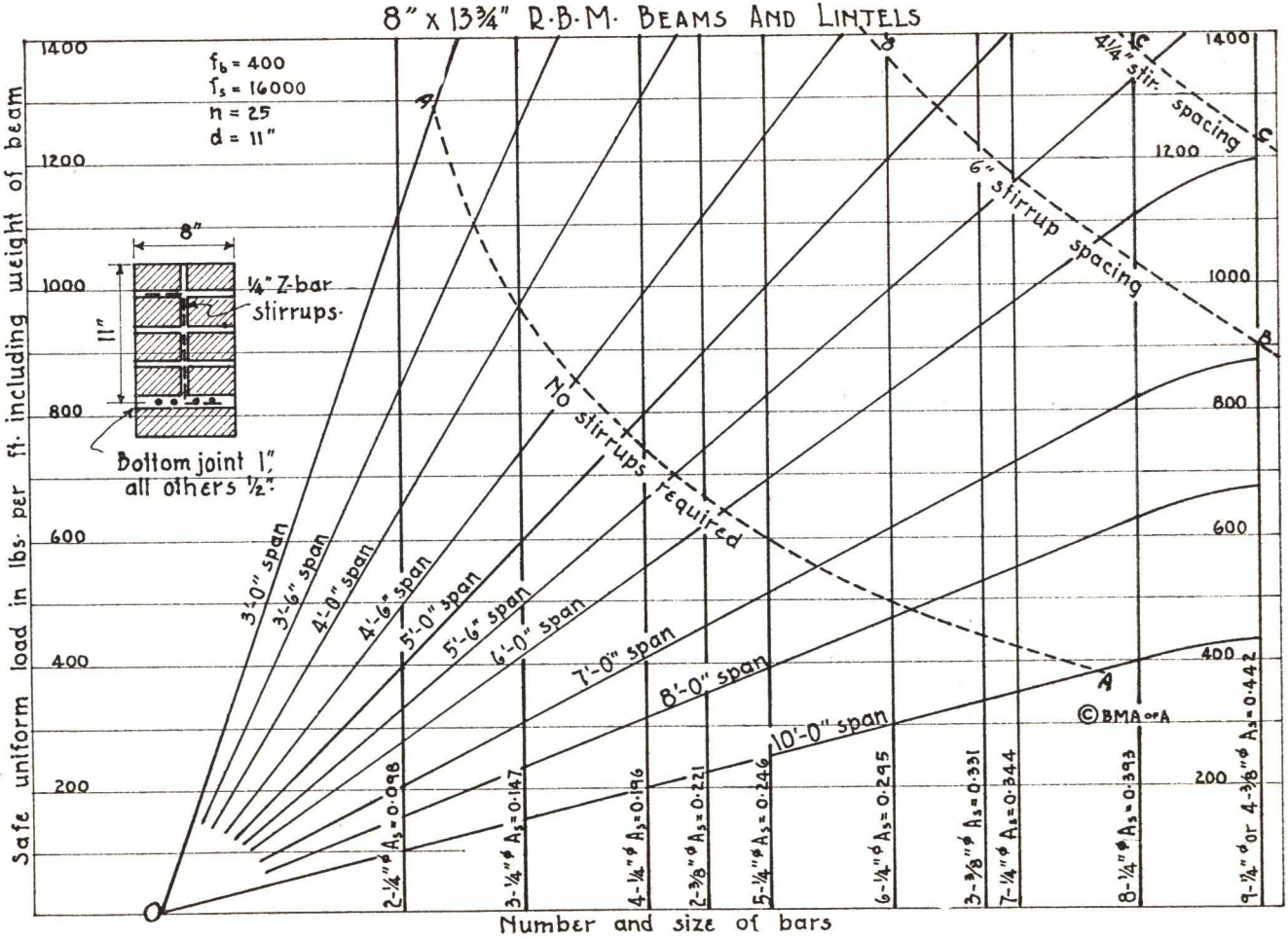

8" x 13¾" R.B.M. Beams and Lintels
$f_b$ = 400
$f_s$ = 16000
n = 25
d = 11"
8"
11"
¼" Z-bar stirrups.
Bottom joint 1"
all others ½".
Safe uniform load in lbs. per ft. including weight of beam
1400
1200
1000
800
600
400
200
3'-0" span
3'-6" span
4'-0" span
4'-6" span
5'-0" span
5'-6" span
6'-0" span
7'-0" span
8'-0" span
10'-0" span
No stirrups required
6" stirrup spacing
4¼" stir. spacing
A
B
C
2-¼"ϕ $A_s$=0.098
3-¼"ϕ $A_s$=0.147
4-¼"ϕ $A_s$=0.196
2-⅜"ϕ $A_s$=0.221
5-¼"ϕ $A_s$=0.246
6-¼"ϕ $A_s$=0.295
3-⅜"ϕ $A_s$=0.331
7-¼"ϕ $A_s$=0.344
8-¼"ϕ $A_s$=0.393
9-¼"ϕ or 4-⅜"ϕ $A_s$=0.442
Number and size of bars
©BMA or A

*Chart No. 1*

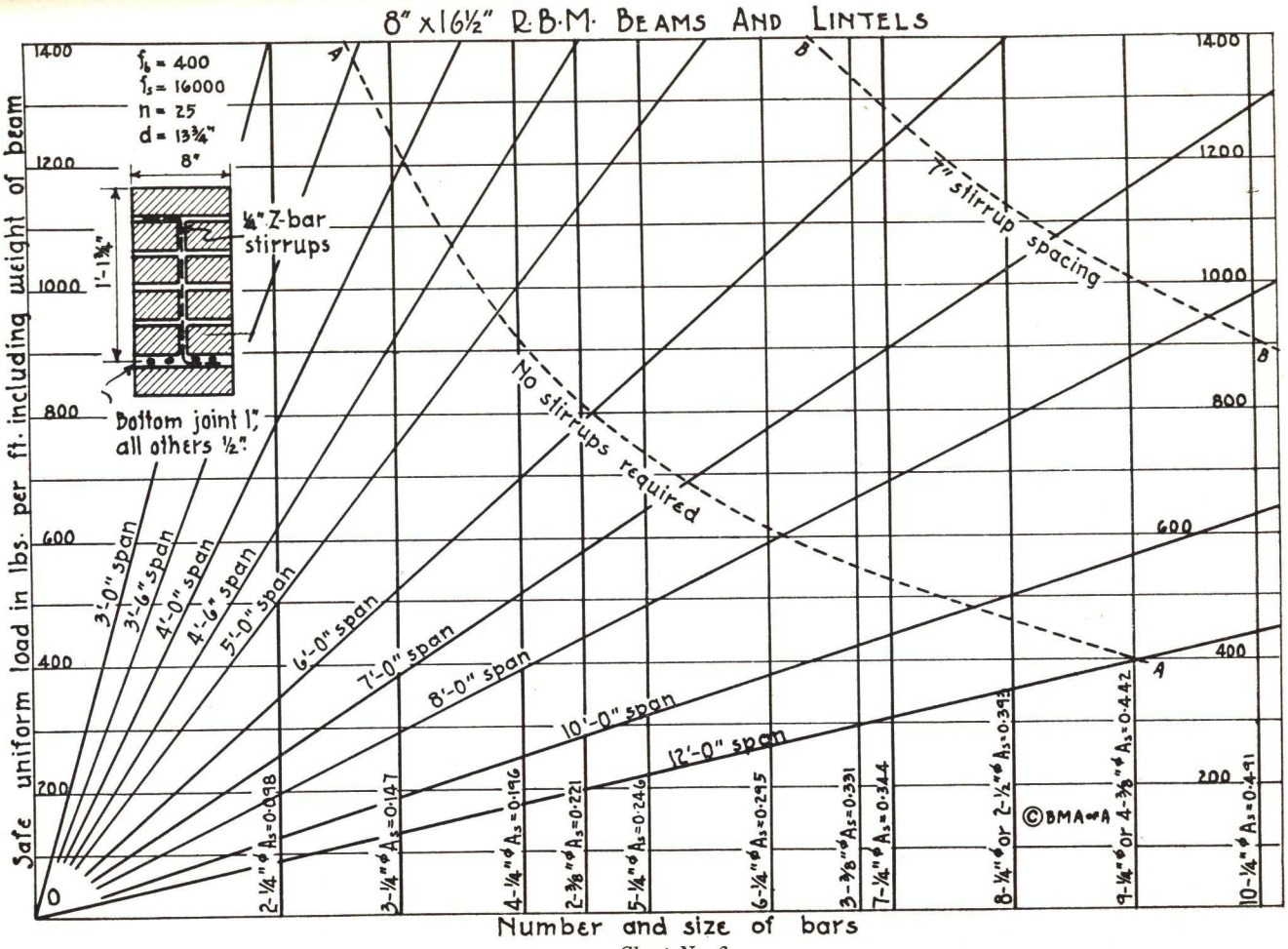

8" x 16½" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
$f_c$ = 400
$f_s$ = 16000
n = 25
d = 13¾"
8"
1'-1¾"
⅛" Z-bar stirrups
Bottom joint 1", all others ½".
No stirrups required
7" stirrup spacing
3'-0" span
3'-6" span
4'-0" span
4'-6" span
5'-0" span
6'-0" span
7'-0" span
8'-0" span
10'-0" span
12'-0" span
2-¼" ϕ As=0.098
3-¼" ϕ As=0.147
4-¼" ϕ As=0.196
2-⅜" ϕ As=0.221
5-¼" ϕ As=0.246
6-¼" ϕ As=0.295
3-⅜" ϕ As=0.331
7-¼" ϕ As=0.344
8-¼" ϕ or 2-½" ϕ As=0.393
9-¼" ϕ or 4-⅜" ϕ As=0.442
10-¼" ϕ As=0.491
©BMA-A
1400
1200
1000
800
600
400
200
0
A
B

*Chart No. 2*

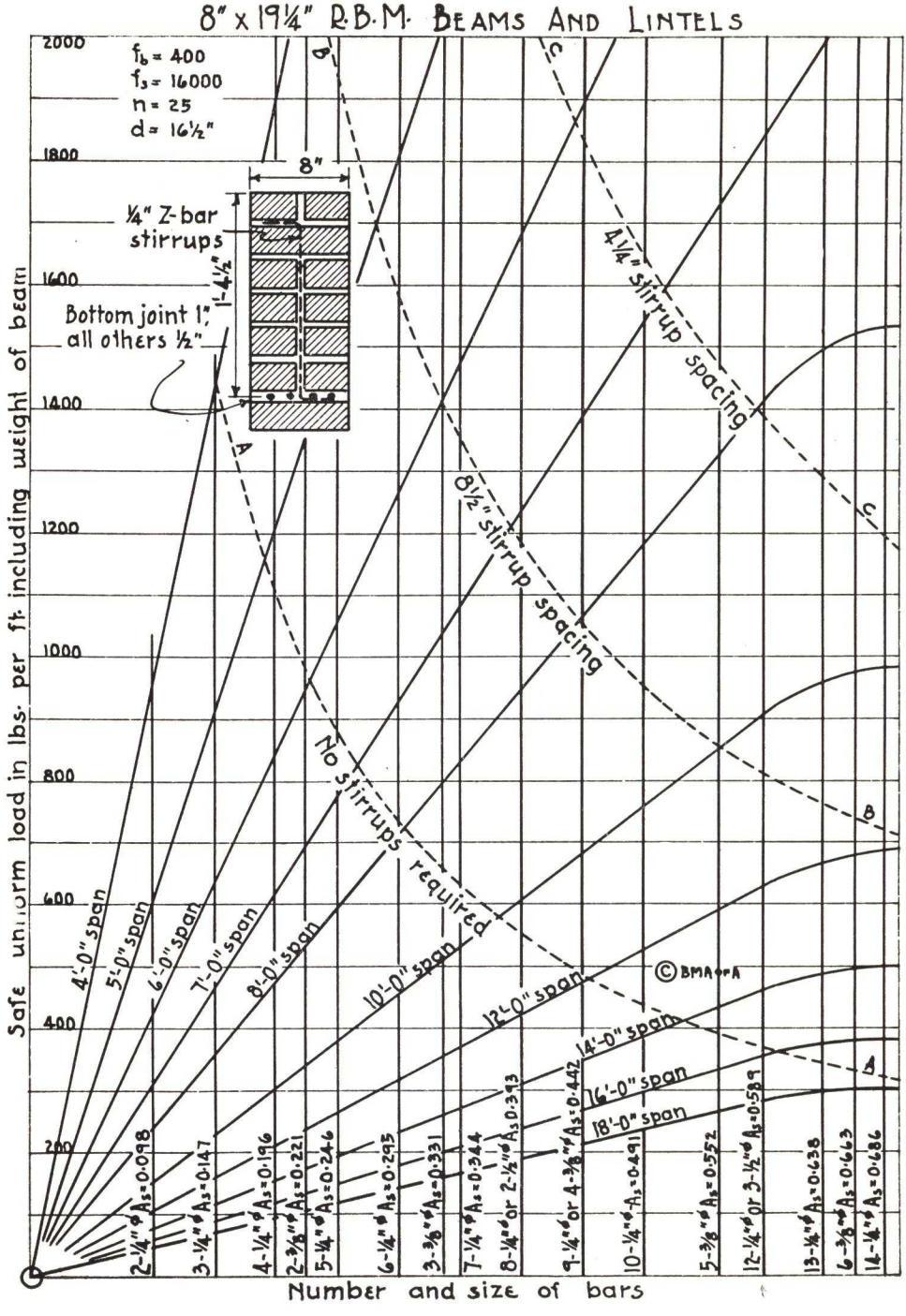

8" x 19¼" R.B.M. Beams and Lintels
$f_b$ = 400
$f_s$ = 16000
n = 25
d = 16½"
8"
¼" Z-bar stirrups
1'-4½"
Bottom joint 1", all others ½"
4¼" stirrup spacing
8½" stirrup spacing
No stirrups required
4'-0" span
5'-0" span
6'-0" span
7'-0" span
8'-0" span
10'-0" span
12'-0" span
14'-0" span
16'-0" span
18'-0" span
© BMA-A
Safe uniform load in lbs. per ft. including weight of beam
2000
1800
1600
1400
1200
1000
800
600
400
200
2-¼"ø As=0.098
3-¼"ø As=0.147
4-¼"ø As=0.196
2-⅜"ø As=0.221
5-¼"ø As=0.246
6-¼"ø As=0.295
3-⅜"ø As=0.331
7-¼"ø As=0.344
8-¼"ø or 2-½"ø As=0.393
9-¼"ø or 4-⅜"ø As=0.442
10-¼"ø As=0.491
5-⅜"ø As=0.552
12-¼"ø or 3-½"ø As=0.589
13-¼"ø As=0.638
6-⅜"ø As=0.663
14-¼"ø As=0.686
Number and size of bars

*Chart No. 3*

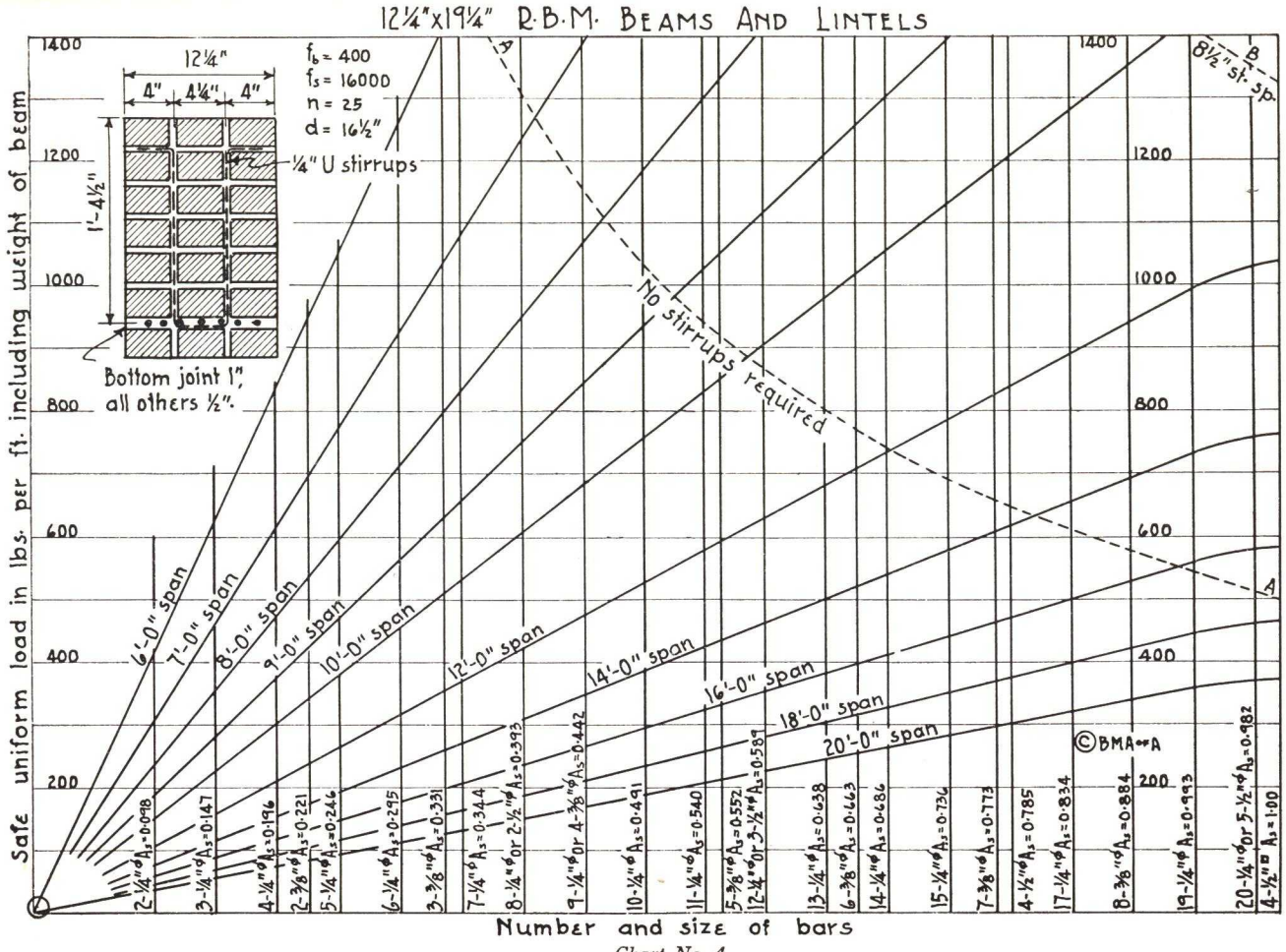

12¼"x19¼" R.B.M. Beams And Lintels
12¼"
4"
4¼"
4"
1'-4½"
$f_b$ = 400
$f_s$ = 16000
n = 25
d = 16½"
¼" U stirrups
Bottom joint 1", all others ½".
No stirrups required
8½" st. sp.
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
6'-0" span
7'-0" span
8'-0" span
9'-0" span
10'-0" span
12'-0" span
14'-0" span
16'-0" span
18'-0" span
20'-0" span
2-¼"ϕ $A_s$=0.098
3-¼"ϕ $A_s$=0.147
4-¼"ϕ $A_s$=0.196
2-⅜"ϕ $A_s$=0.221
5-¼"ϕ $A_s$=0.246
6-¼"ϕ $A_s$=0.295
3-⅜"ϕ $A_s$=0.331
7-¼"ϕ $A_s$=0.344
8-¼"ϕ or 2-½"ϕ $A_s$=0.393
9-¼"ϕ or 4-⅜"ϕ $A_s$=0.442
10-¼"ϕ $A_s$=0.491
11-¼"ϕ $A_s$=0.540
5-⅜"ϕ $A_s$=0.552
12-¼"ϕ or 3-½"ϕ $A_s$=0.589
13-¼"ϕ $A_s$=0.638
6-⅜"ϕ $A_s$=0.663
14-¼"ϕ $A_s$=0.686
15-¼"ϕ $A_s$=0.736
7-⅜"ϕ $A_s$=0.773
4-½"ϕ $A_s$=0.785
17-¼"ϕ $A_s$=0.834
8-⅜"ϕ $A_s$=0.884
19-¼"ϕ $A_s$=0.933
20-¼"ϕ or 5-½"ϕ $A_s$=0.982
4-½"□ $A_s$=1.00
©BMA-A

*Chart No. 4*

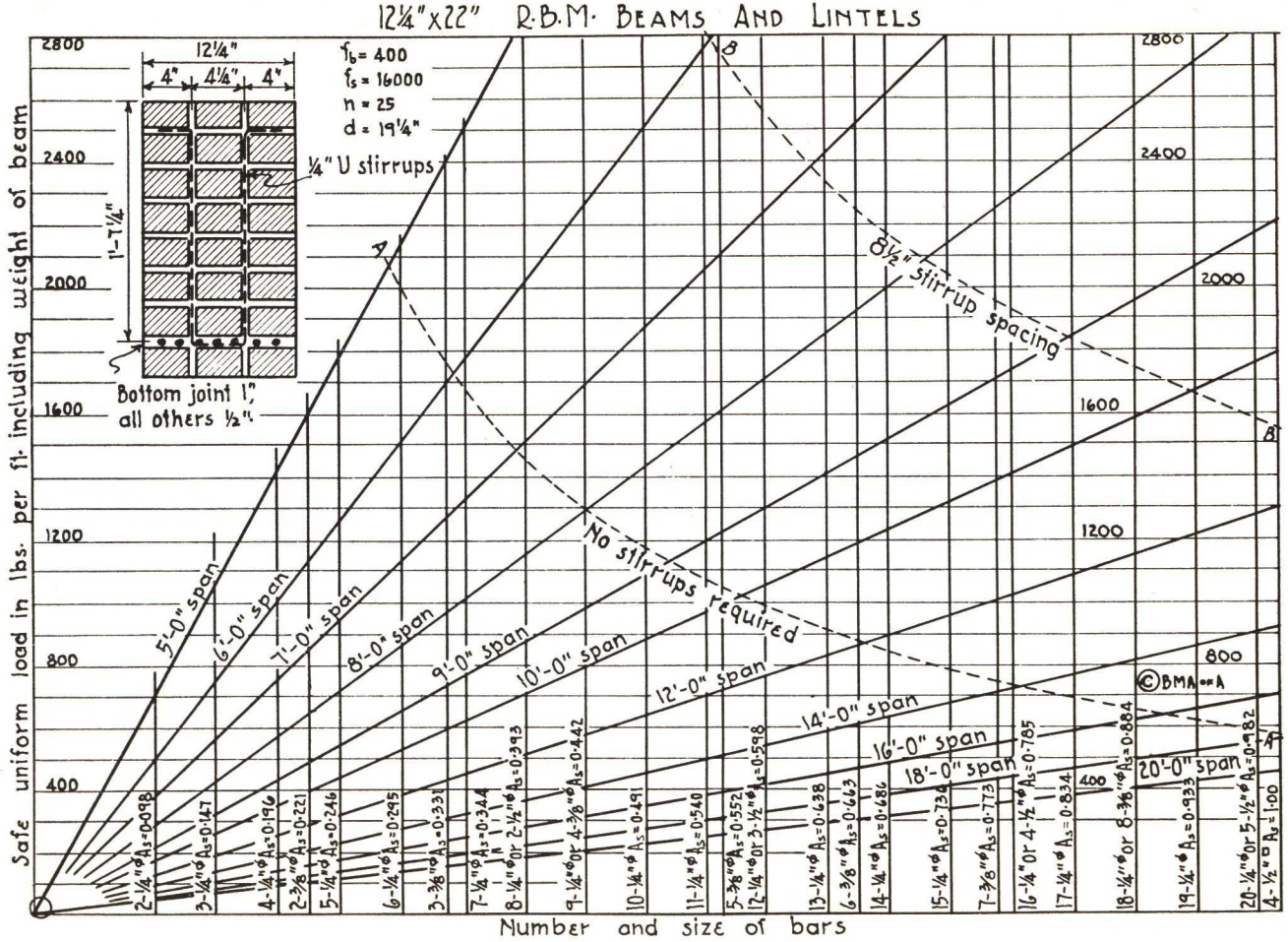

12¼"x22" R.B.M. Beams And Lintels
12¼"
4"
4¼"
4"
1'-7¼"
Bottom joint 1", all others ½"
$f_b$ = 400
$f_s$ = 16000
n = 25
d = 19¼"
¼" U stirrups
8½" stirrup spacing
No stirrups required
5'-0" span
6'-0" span
7'-0" span
8'-0" span
9'-0" span
10'-0" span
12'-0" span
14'-0" span
16'-0" span
18'-0" span
20'-0" span
2-¼"ø As=0·098
3-¼"ø As=0·147
4-¼"ø As=0·196
2-⅜"ø As=0·221
5-¼"ø As=0·246
6-¼"ø As=0·295
3-⅜"ø As=0·331
7-¼"ø As=0·344
8-¼"ø or 2-½"ø As=0·393
9-¼"ø or 4-⅜"ø As=0·442
10-¼"ø As=0·491
11-¼"ø As=0·540
5-⅜"ø As=0·552
12-¼"ø or 3-½"ø As=0·589
13-¼"ø As=0·638
6-⅜"ø As=0·663
14-¼"ø As=0·687
15-¼"ø As=0·736
7-⅜"ø As=0·773
16-¼" or 4-½"ø As=0·785
17-¼"ø As=0·834
18-¼"ø or 8-⅜"ø As=0·884
19-¼"ø As=0·933
20-¼"ø or 5-½"ø As=0·982
4-½"□ As=1·00
©BMA-A
Safe uniform load in lbs. per ft. including weight of beam
2800
2400
2000
1600
1200
800
400
Number and size of bars

*Chart No. 5*

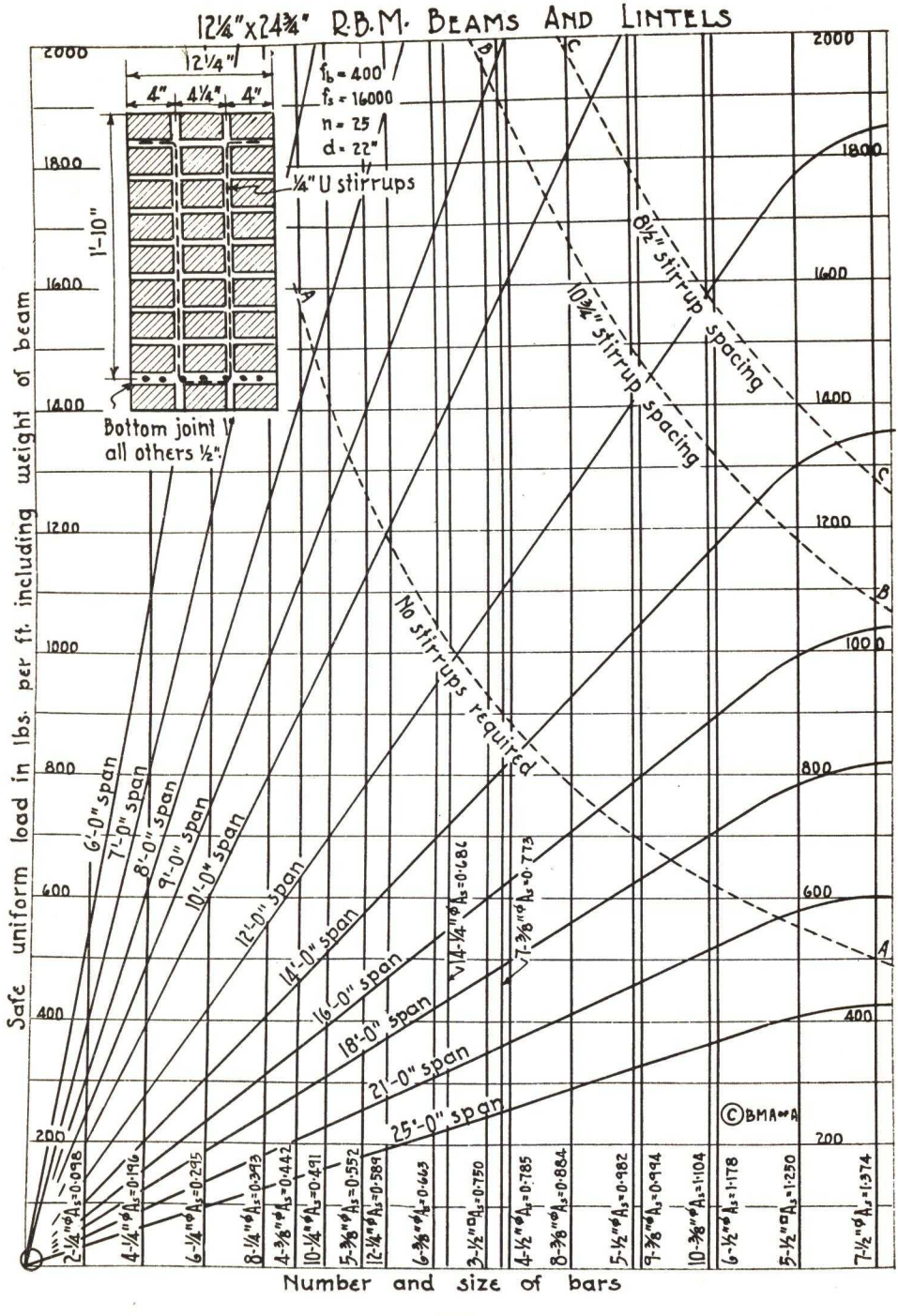

12¼"x24¾" R.B.M. Beams and Lintels
12¼"
4"
4¼"
4"
1'-10"
Bottom joint ½"
all others ½".
fb = 400
fs = 16000
n = 25
d = 22"
¼" U stirrups
8½" stirrup spacing
10¾" stirrup spacing
No stirrups required
6'-0" span
7'-0" span
8'-0" span
9'-0" span
10'-0" span
12'-0" span
14'-0" span
16'-0" span
18'-0" span
21'-0" span
25'-0" span
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
2000
1800
1600
1400
1200
1000
800
600
400
200
2-¼"ϕ As=0.098
4-¼"ϕ As=0.196
6-¼"ϕ As=0.295
8-¼"ϕ As=0.393
4-⅜"ϕ As=0.442
10-¼"ϕ As=0.491
5-⅜"ϕ As=0.552
12-¼"ϕ As=0.589
6-⅜"ϕ As=0.663
3-½"ϕ As=0.750
4-½"ϕ As=0.785
8-⅜"ϕ As=0.884
5-½"ϕ As=0.982
9-⅜"ϕ As=0.994
10-⅜"ϕ As=1.104
6-½"ϕ As=1.178
5-½"ϕ As=1.250
7-½"ϕ As=1.374
1-1¼"ϕ As=0.686
1-⅜"ϕ As=0.773
©BMA-A

*Chart No. 6*

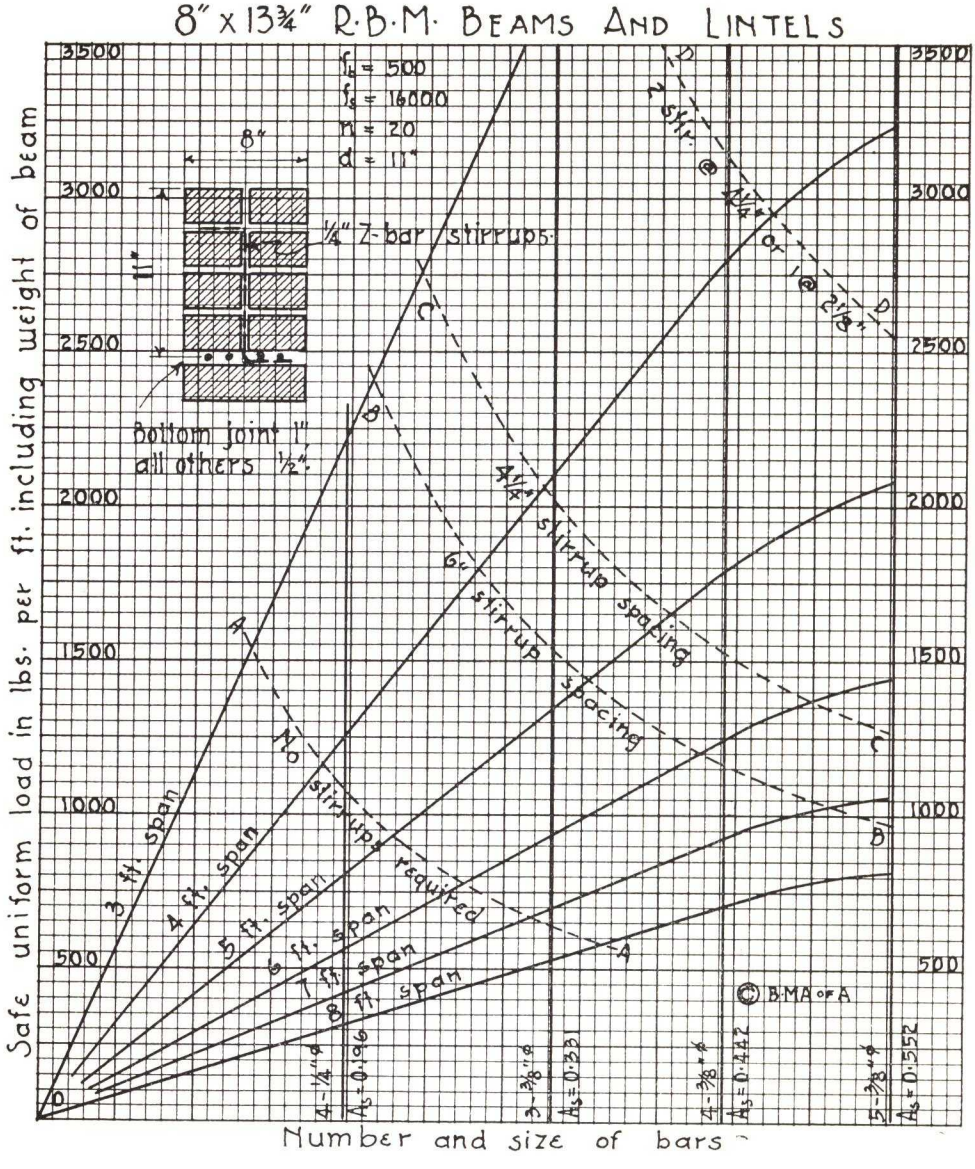

8" x 13¾" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
3500
3000
2500
2000
1500
1000
500
0
$f_c$ = 500
$f_s$ = 16000
n = 20
d = 11"
8"
11"
¼" Z-bar stirrups
Bottom joint 1"
all others ½".
3 ft. span
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
stirrup required
4½" stirrup spacing
6" stirrup spacing
2 stir. @ 4½" & 6" & 2½"
4-¼" ϕ
As = 0.196
3-⅜" ϕ
As = 0.331
4-⅜" ϕ
As = 0.442
5-⅜" ϕ
As = 0.552
© B.M.A. of A.

*Chart No. 7*

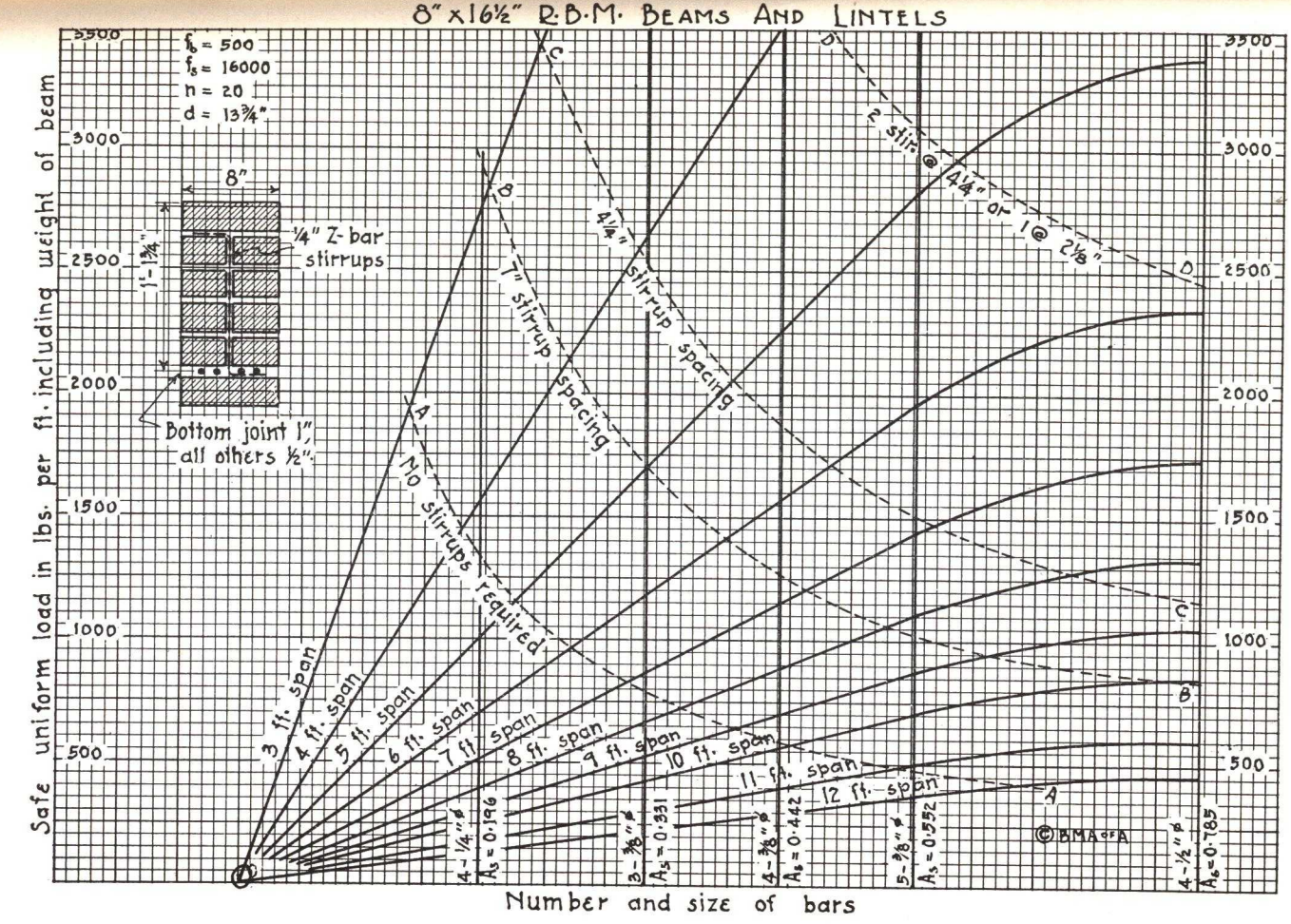

8" x 16½" R.B.M. BEAMS AND LINTELS
$f_c$ = 500
$f_s$ = 16000
n = 20
d = 13¾"
8"
1'-1¾"
¼" Z-bar stirrups
Bottom joint 1", all others ½".
Safe uniform load in lbs. per ft. including weight of beam
3500
3000
2500
2000
1500
1000
500
A
B
C
D
No stirrups required
7" stirrup spacing
4¼" stirrup spacing
2 stir. @ 4¼" or 1 @ 2⅛"
3 ft. span
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
11 ft. span
12 ft. span
4-¼" ϕ
$A_s$ = 0.196
3-⅜" ϕ
$A_s$ = 0.331
4-⅜" ϕ
$A_s$ = 0.442
5-⅜" ϕ
$A_s$ = 0.552
4-½" ϕ
$A_s$ = 0.785
© BMA of A
Number and size of bars

*Chart No. 8*

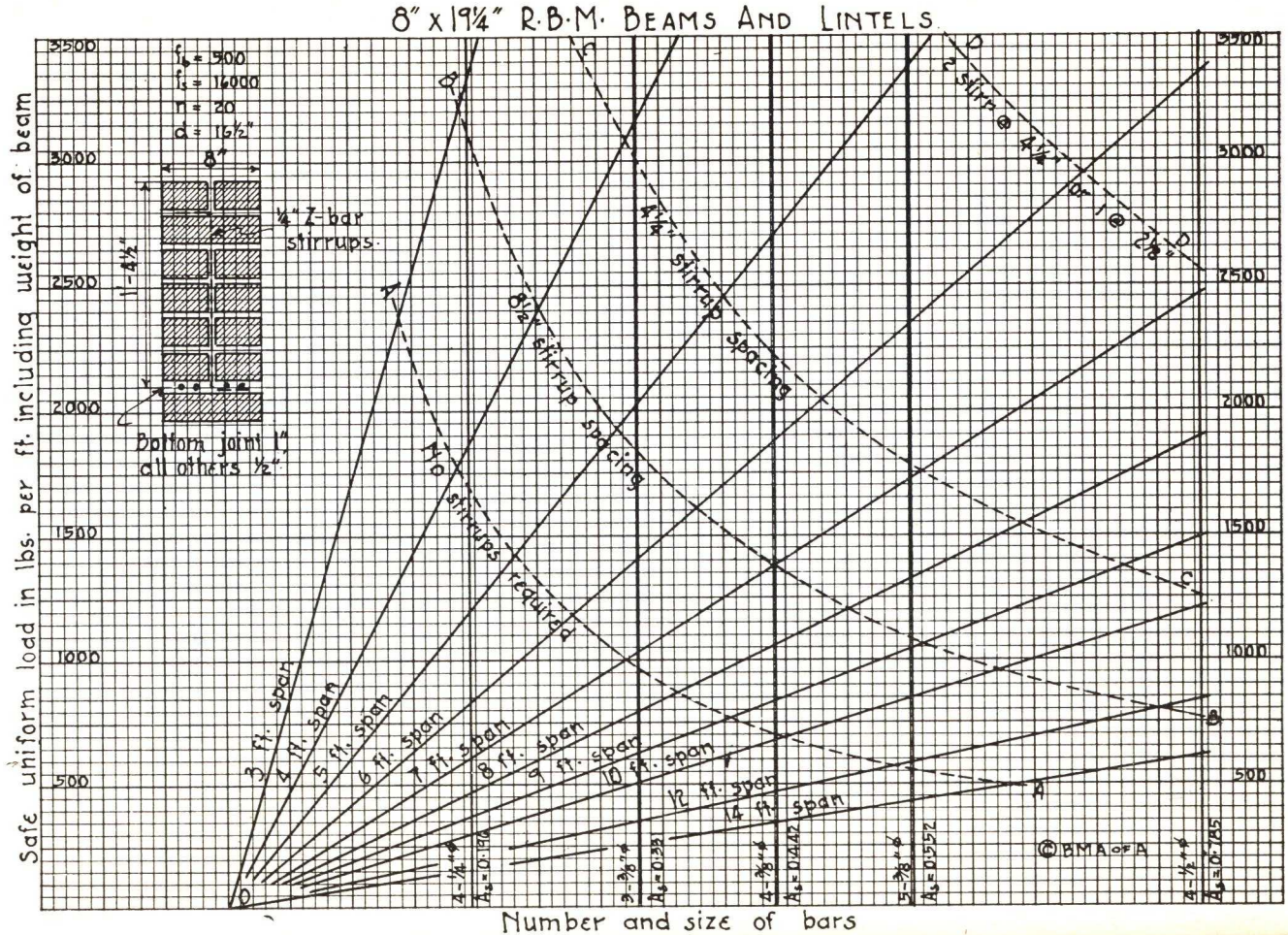

8" x 19¼" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
$f_c$ = 500
$f_s$ = 16000
n = 20
d = 16½"
8"
¼" Z-bar stirrups
11-4½"
Bottom joint 1", all others ½"
3 ft. span
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
No stirrups required
6" stirrup spacing
4½" stirrup spacing
2 stirrups @ 4½"
2 stirrups @ 2½"
4-¼"ϕ $A_s$=0.196
3-⅜"ϕ $A_s$=0.331
4-⅜"ϕ $A_s$=0.4420
5-⅜"ϕ $A_s$=0.5525
4-½"ϕ $A_s$=0.785
BMA of P
35500
30000
25000
20000
15000
10000
5000
A
B
C
D
E
F

*Chart No. 9*

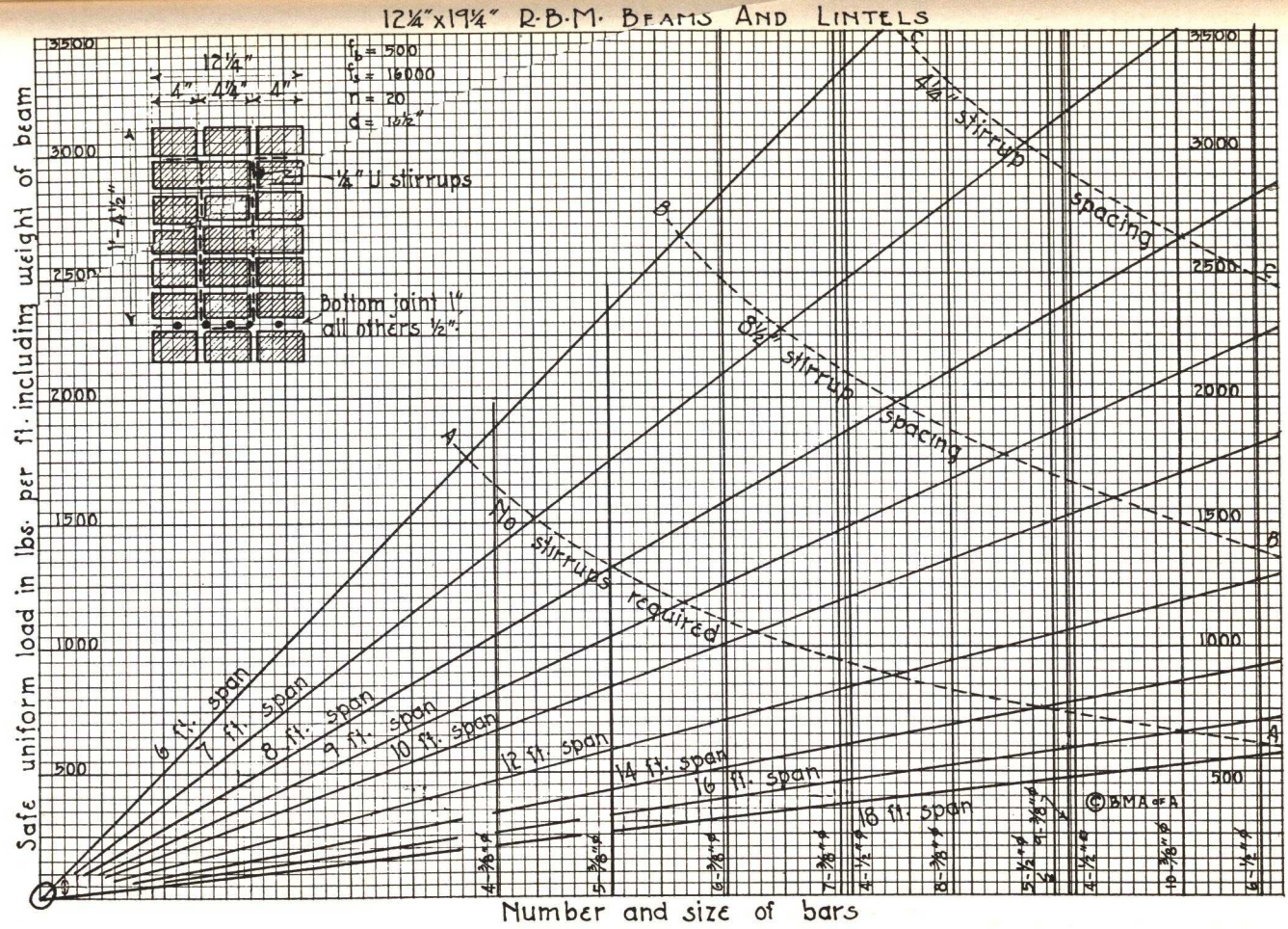

12¼"x19¼" R·B·M· Beams And Lintels
$f_b$ = 500
$f_s$ = 16000
n = 20
d = 16½"
12¼"
4" 4¼" 4"
1'-4½"
¼" U stirrups
Bottom joint ½"
all others ½"
4½" stirrup spacing
8½" stirrup spacing
No stirrups required
A
B
C
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
Safe uniform load in lbs. per ft. including weight of beam
35000
30000
25000
20000
15000
10000
500
4-⅜"ø
5-⅜"ø
6-⅜"ø
7-⅜"ø
4-½"ø
8-⅜"ø
5-½"ø
9-⅜"ø
4-½"ø
10-⅜"ø
6-½"ø
©BMA-A
Number and size of bars

*Chart No. 10*

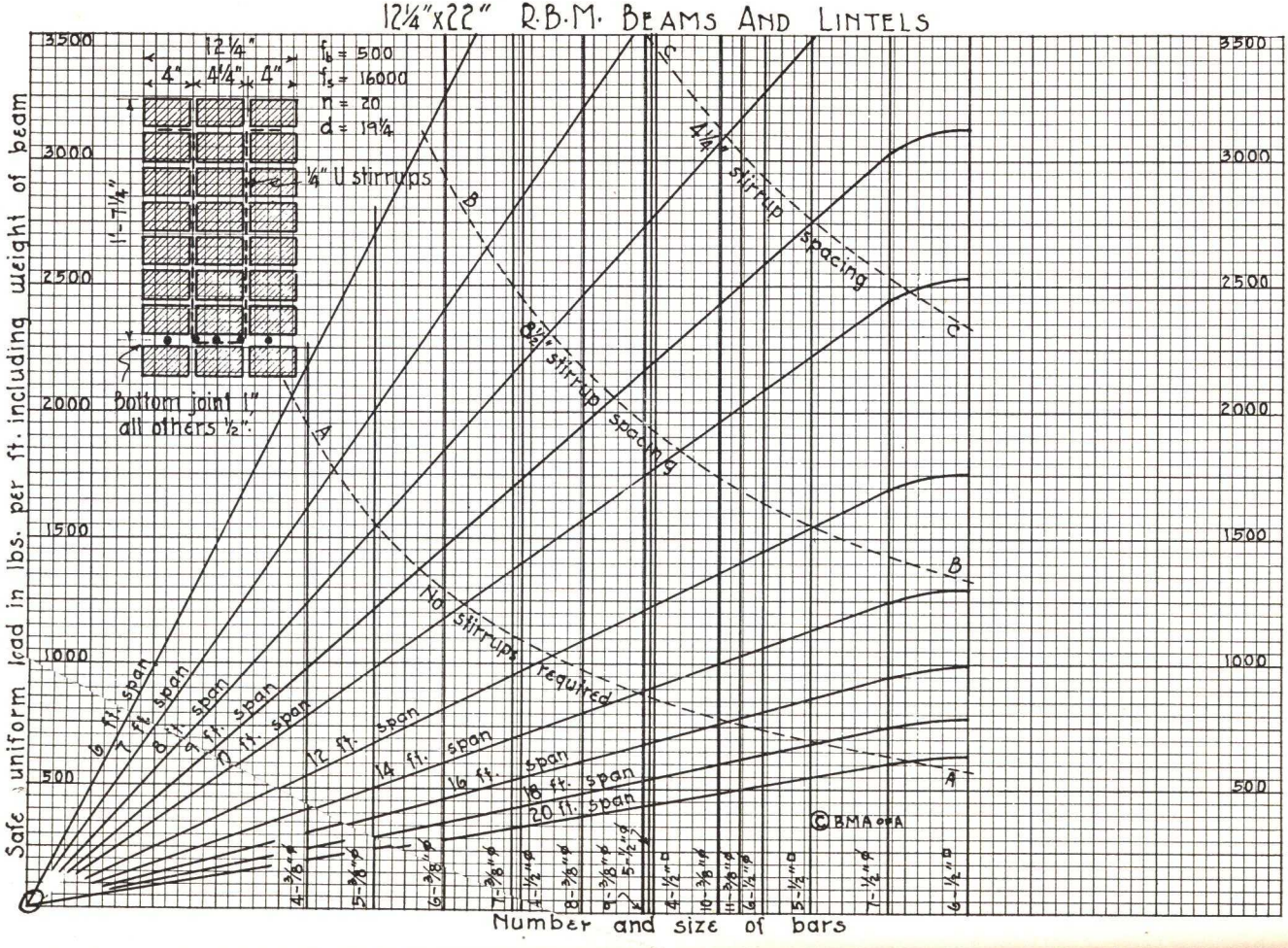



*Chart No. 11*

12¼"x24¾" R.B.M. Beams And Lintels

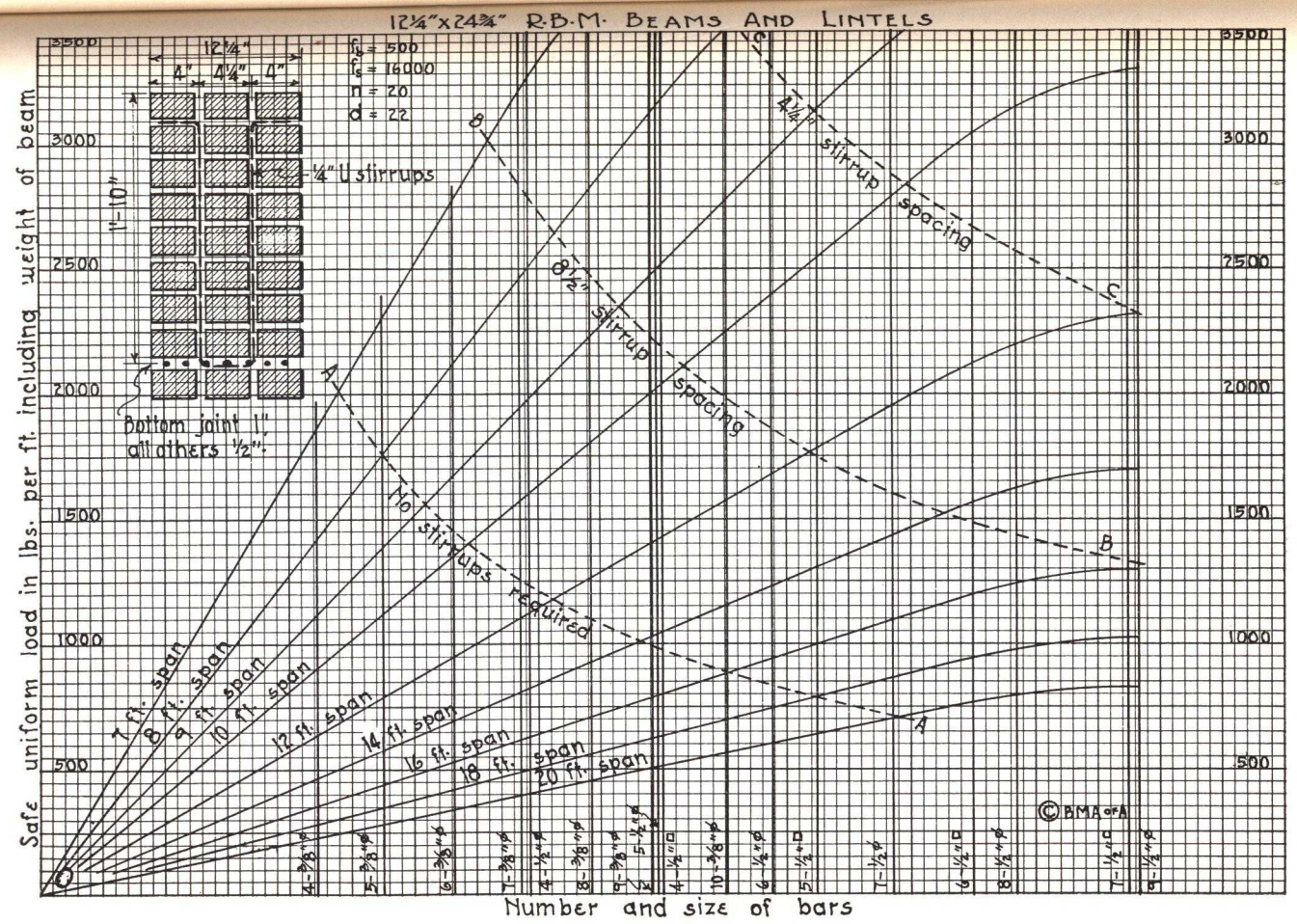

12¼"x24¾" R.B.M. Beams and Lintels
$f_s$ = 500
$f_c$ = 16000
n = 20
d = 22
12¼"
4"
4¼"
4"
¼" = ¼" U stirrups
11"-10"
Bottom joint 1"
all others ½".
Safe uniform load in lbs. per ft. including weight of beam
3500
3000
2500
2000
1500
1000
500
A
B
C
8½" Stirrup spacing
4¼" Stirrup spacing
No stirrups required
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
20 ft. span
4-⅜"ø
5-⅜"ø
6-⅜"ø
7-⅜"ø
4-½"ø
8-⅜"ø
9-⅜"ø
5-½"ø
4-¼"□
10-⅜"ø
6-½"ø
5-½"□
7-½"ø
6-¼"□
8-½"ø
7-½"□
9-½"ø
© BMA of A
Number and size of bars

*Chart No. 12*

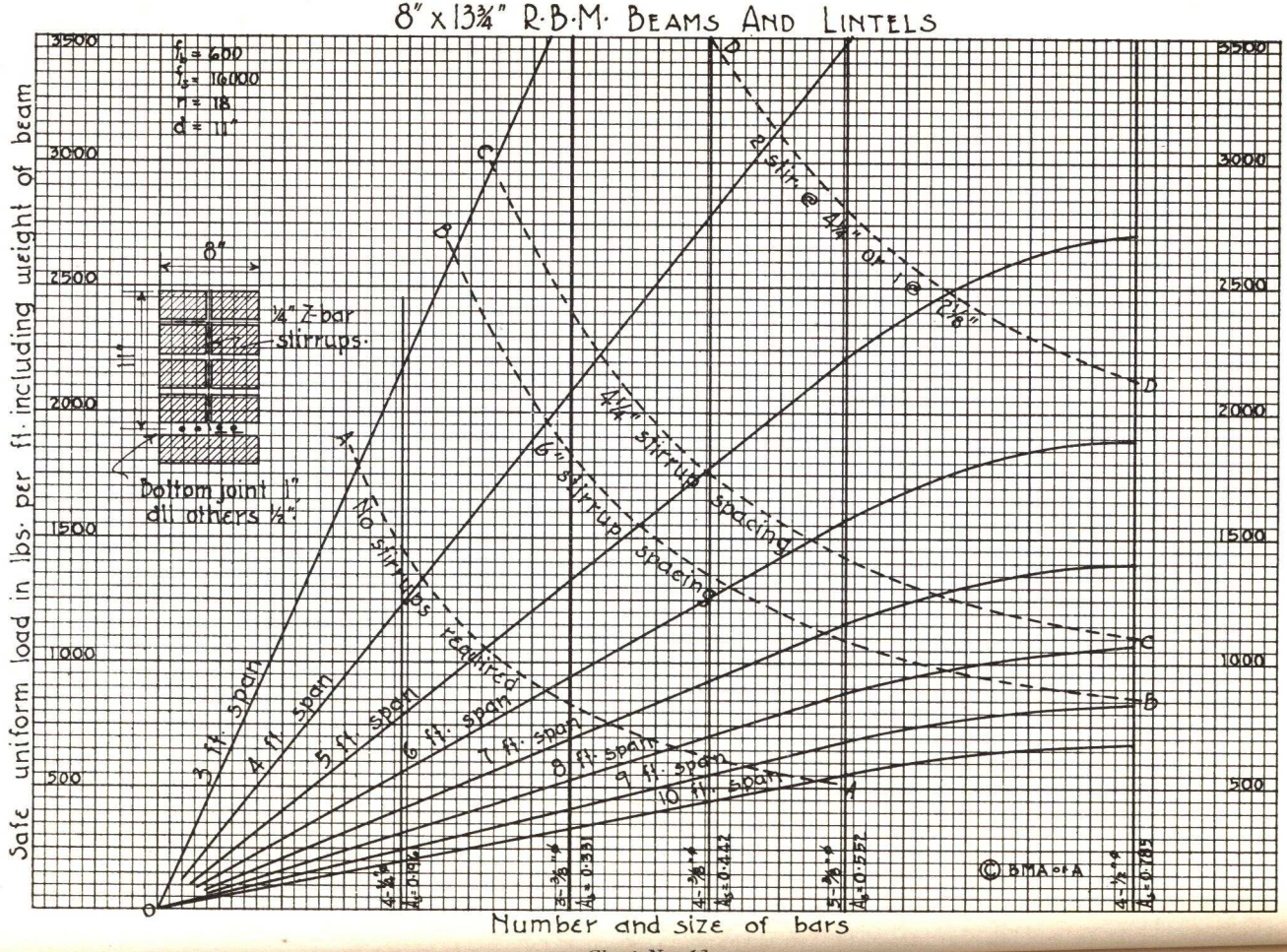



*Chart No. 13*

8" x 14¼" R.B.M. Beams And Lintels

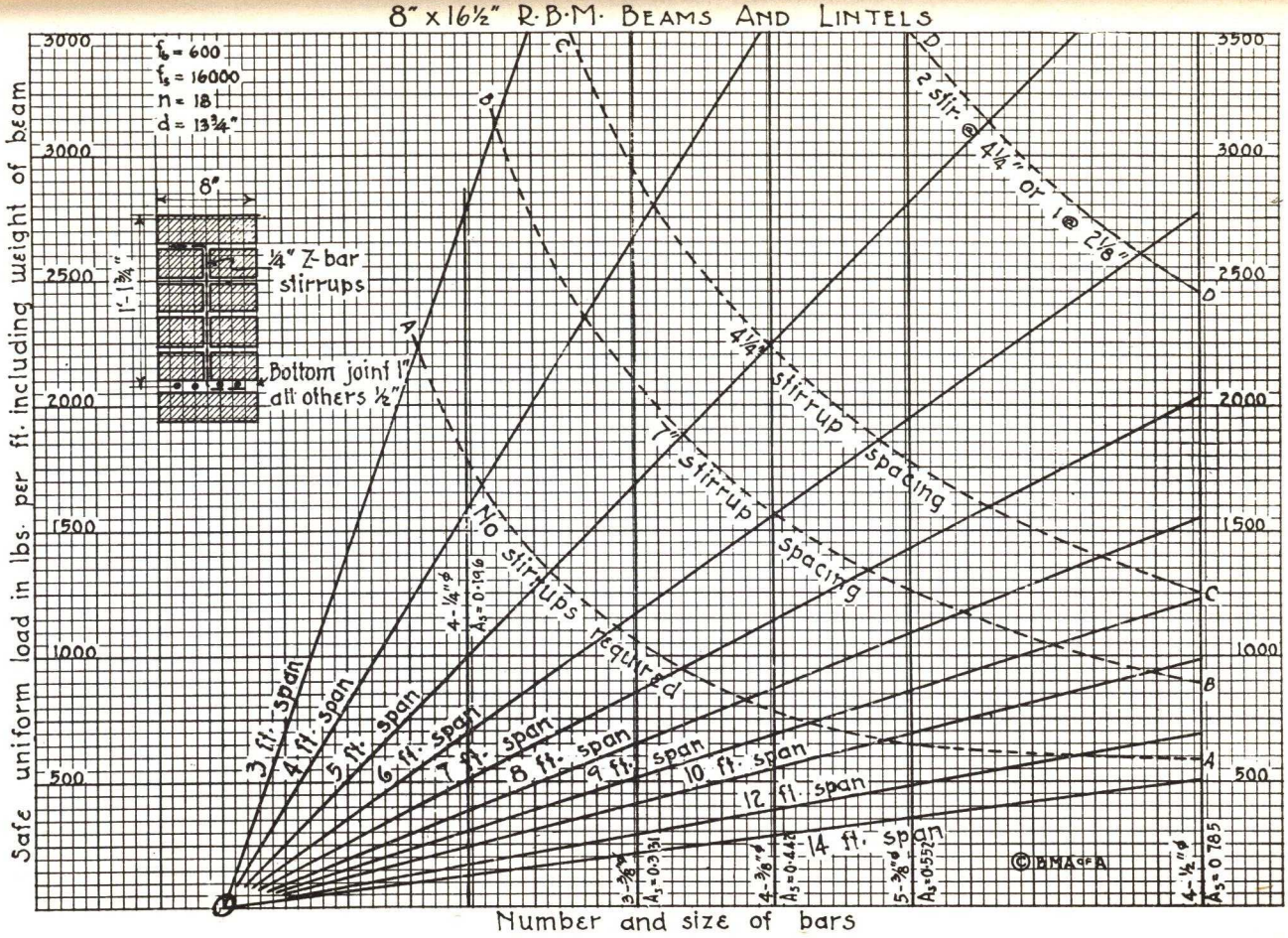

8" x 16½" R.B.M. Beams And Lintels
$f_b$ = 600
$f_s$ = 16000
n = 18
d = 13¾"
8"
1'-1¾"
¼" Z-bar stirrups
Bottom joint 1" all others ½"
Safe uniform load in lbs. per ft. including weight of beam
3000
2500
2000
1500
1000
500
3500
A
B
C
D
2 stir. @ 4¼" or 1 @ 2⅛"
4¼" stirrup spacing
7" stirrup spacing
No stirrups required
3 ft. span
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
4-¼"ø As=0.196
3-⅜"ø As=0.331
4-⅜"ø As=0.442
5-⅜"ø As=0.552
4-½"ø As=0.785
©BMA of A
Number and size of bars

*Chart No. 14*

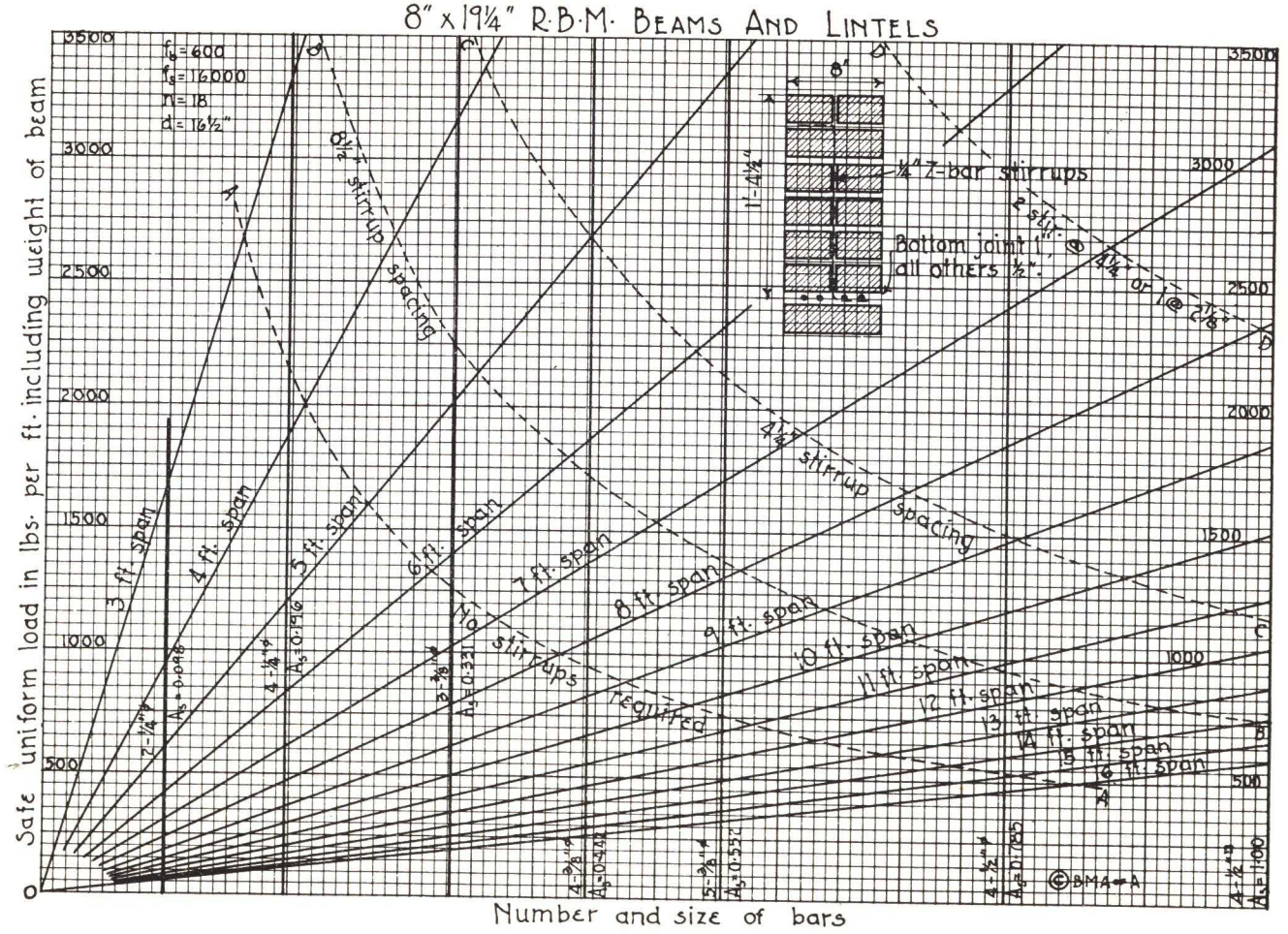

8" x 19¼" R.B.M. Beams And Lintels
$f_c$ = 600
$f_s$ = 16000
n = 18
d = 16½"
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
3 ft. span
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
11 ft. span
12 ft. span
13 ft. span
14 ft. span
15 ft. span
16 ft. span
6" stirrup spacing
4½" stirrup spacing
3" stirrup spacing
stirrups required
¼" Z-bar stirrups
Bottom joint 1", all others ½".
2 sets @ 4¼" or 1 @ 2⅛"
11-4½"
8"
$A_s$=0.046
$A_s$=0.196
$A_s$=0.331
$A_s$=0.442
$A_s$=0.552
$A_s$=0.765
$A_s$=1.100
© BMA

*Chart No. 15*

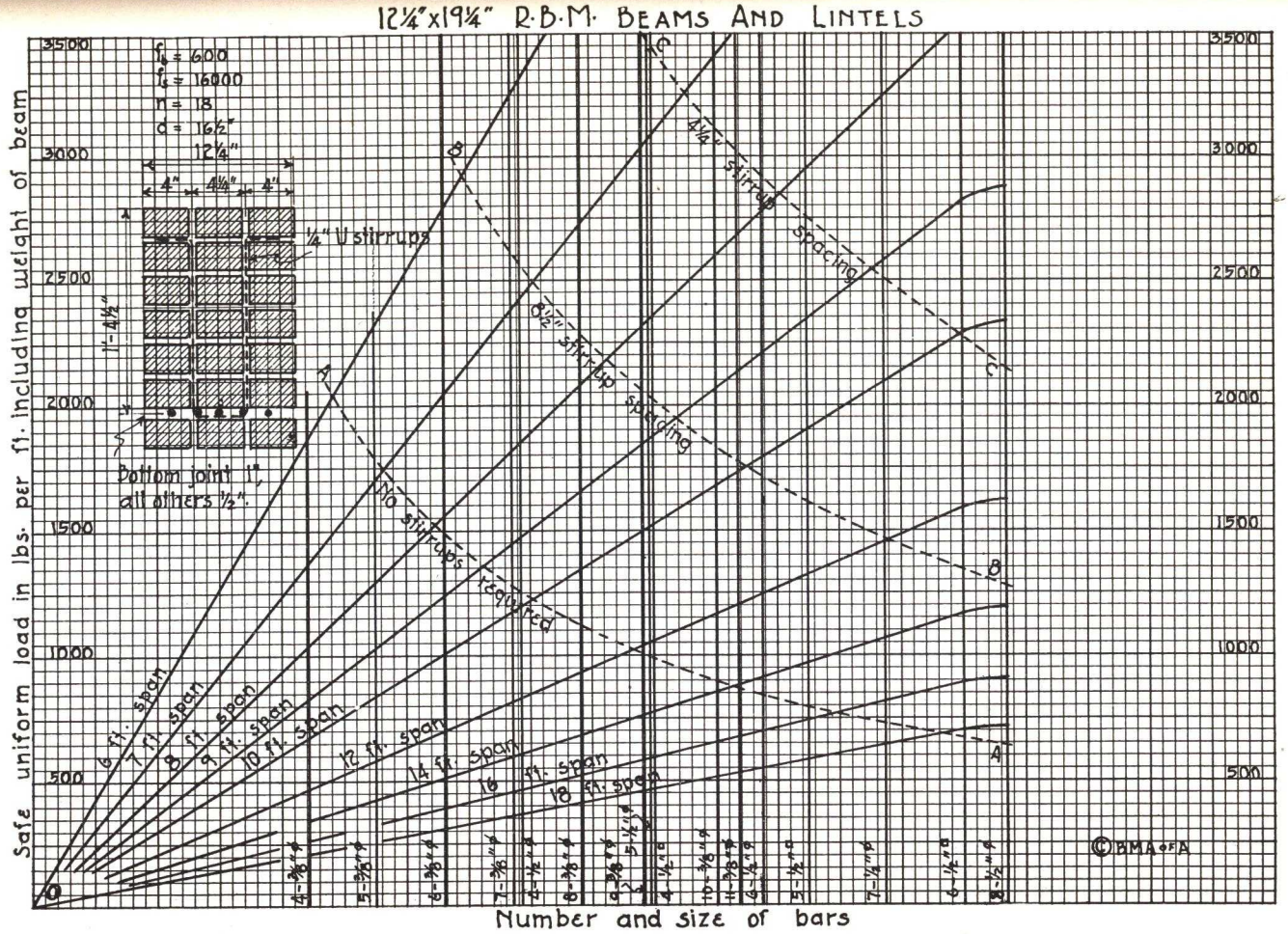

12¼"x19¼" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
$f_c$ = 600
$f_s$ = 16000
n = 18
d = 16½"
12¼"
¼" U stirrups
Bottom joint 1", all others ½".
No stirrups required
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
4¼" stirrup spacing
6½" stirrup spacing
©BMA-EA

*Chart No. 16*

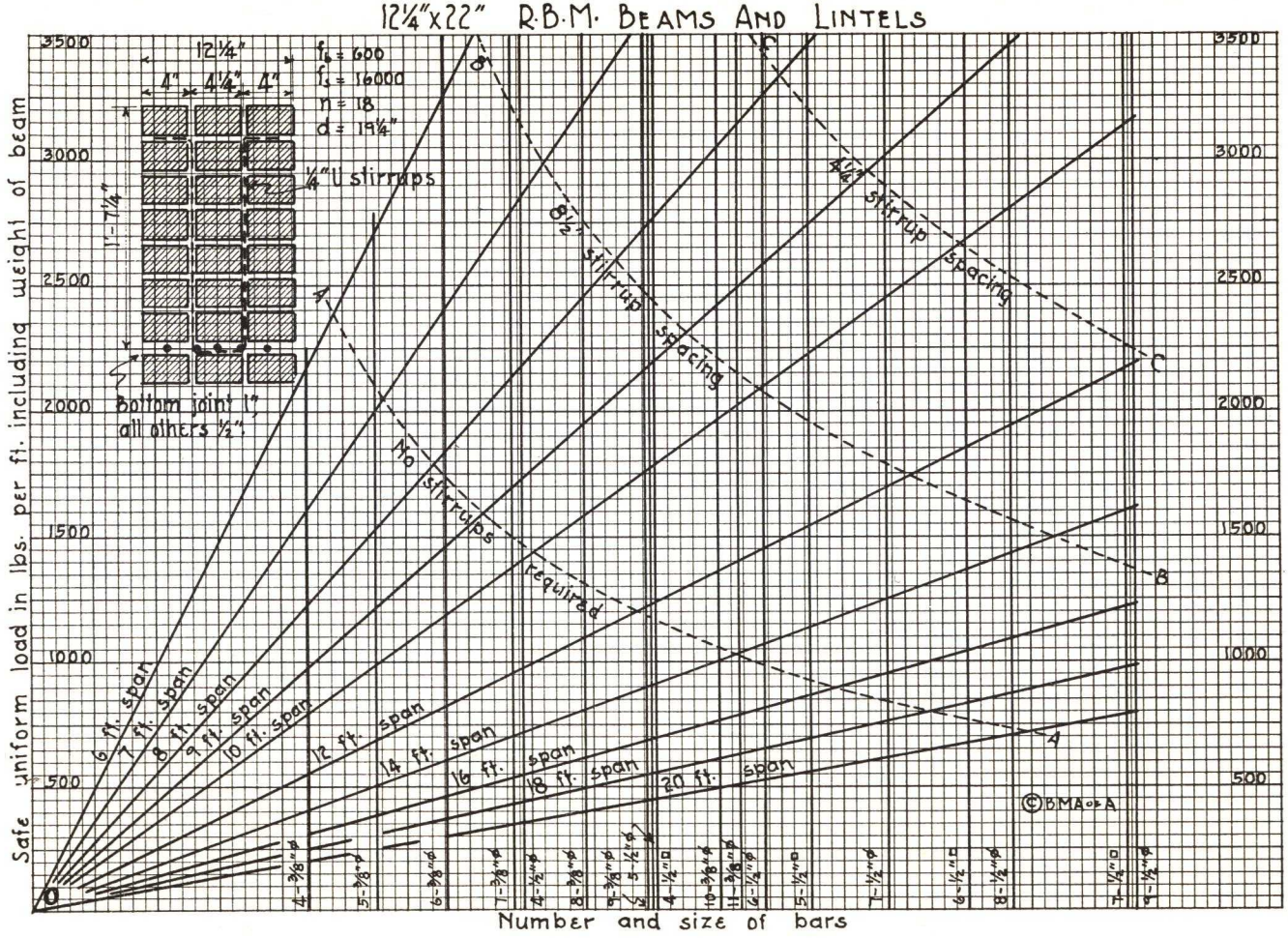

12¼"x22" R.B.M. Beams And Lintels
12¼"
4" 4¼" 4"
1'-7¼"
Bottom joint 1"
all others ½"
$f_s$ = 600
$f_c$ = 16000
n = 18
d = 19¼"
¼" U stirrups
Safe uniform load in lbs. per ft. including weight of beam
3500
3000
2500
2000
1500
1000
500
0
A
B
C
D
E
F
8" stirrup spacing
8½" stirrup spacing
4½" stirrup spacing
No stirrups required
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
20 ft. span
©BMA-A
4-⅜"ø
5-⅜"ø
6-⅜"ø
7-⅜"ø
4-½"ø
8-⅜"ø
9-⅜"ø
5-½"ø
4-½"□
10-⅜"ø
11-⅜"ø
6-½"ø
5-½"ø
7-½"ø
6-½"□
8-½"ø
7-¼"□
9-½"ø
Number and size of bars

*Chart No. 17*

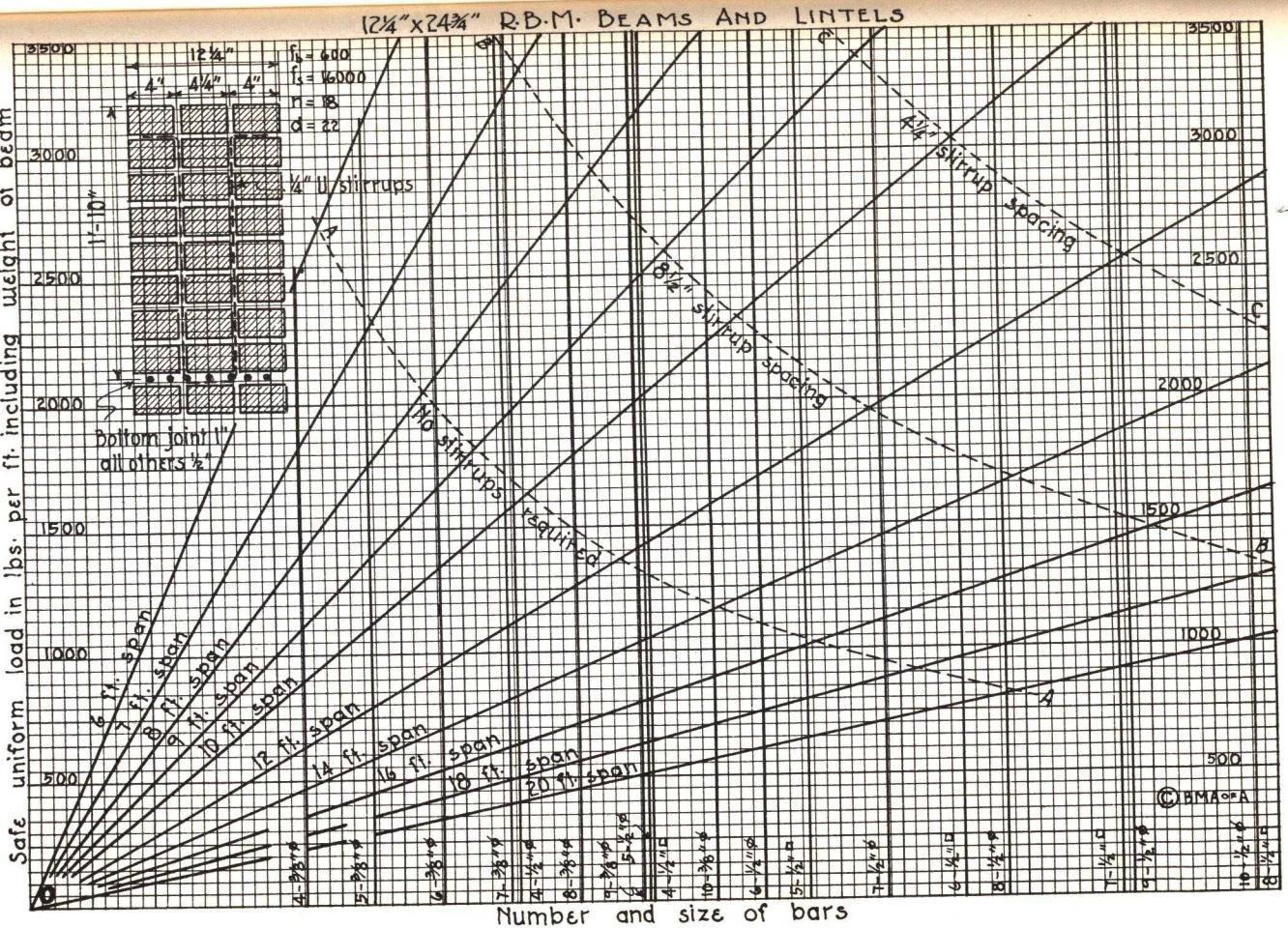

12¼" x 24¾" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
12¼"
4"
4¼"
4"
11'-10"
Bottom joint 1" all others ½"
¼" U stirrups
$f_b$ = 600
$f_s$ = 16000
n = 18
d = 22
No stirrups required
4¼" stirrup spacing
8¼" stirrup spacing
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
20 ft. span
©BMA of A

*Chart No. 18*

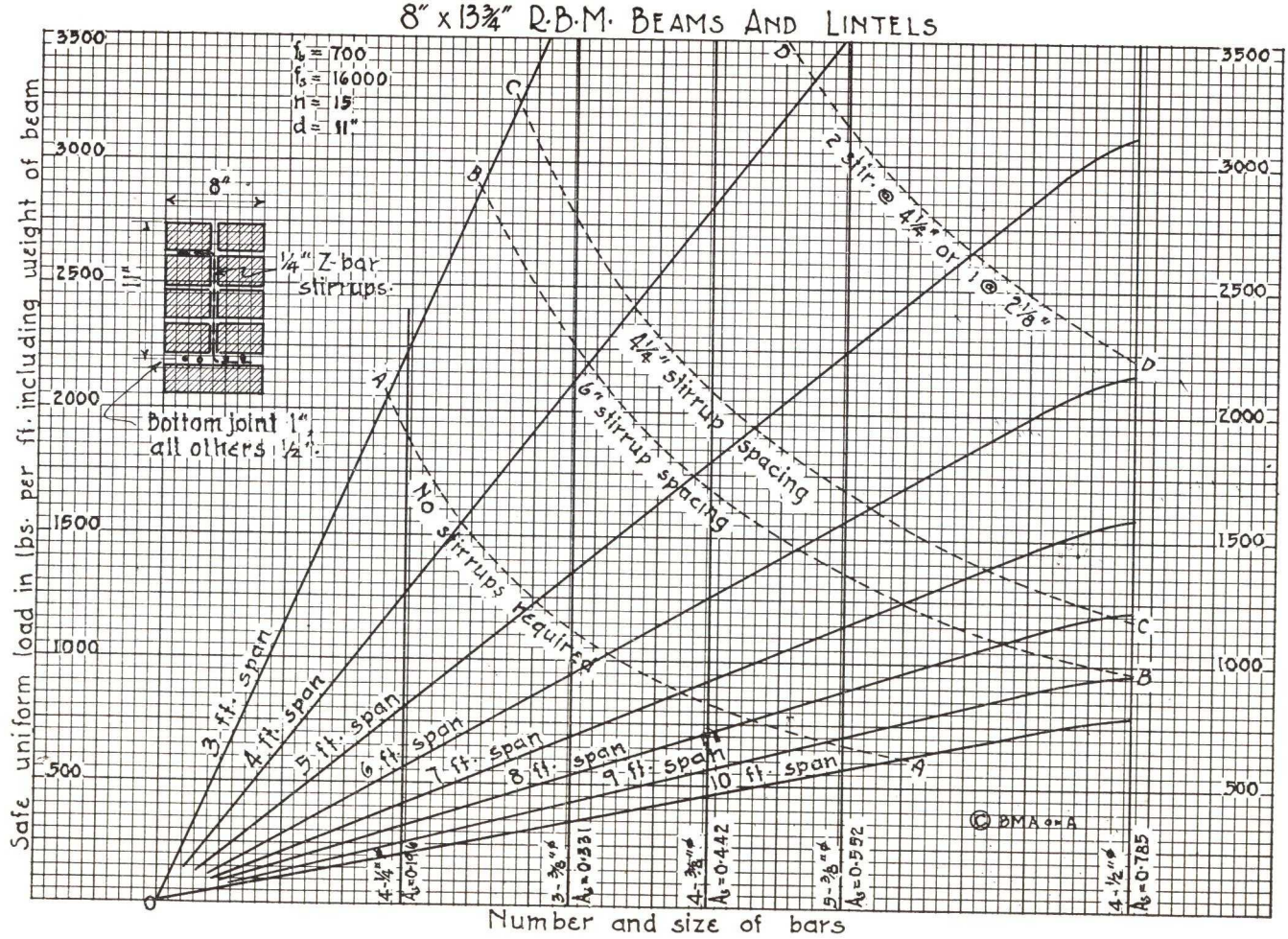

8" x 13¾" R.B.M. Beams And Lintels
$f_c$ = 700
$f_s$ = 16000
n = 15
d = 11"
8"
11"
¼" Z-bar stirrups
Bottom joint 1" all others ½"
Safe uniform load in lbs. per ft. including weight of beam
3500
3000
2500
2000
1500
1000
500
3 ft. span
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
No stirrups required
6" stirrup spacing
4¼" stirrup spacing
2 stir. @ 4¼" or 1 @ 2⅛"
A
B
C
D
4-¼" $A_s$=0.196
3-⅜" $A_s$=0.331
4-⅜" $A_s$=0.442
5-⅜" $A_s$=0.552
4-½" $A_s$=0.785
© DMA & A
Number and size of bars

*Chart No. 19*

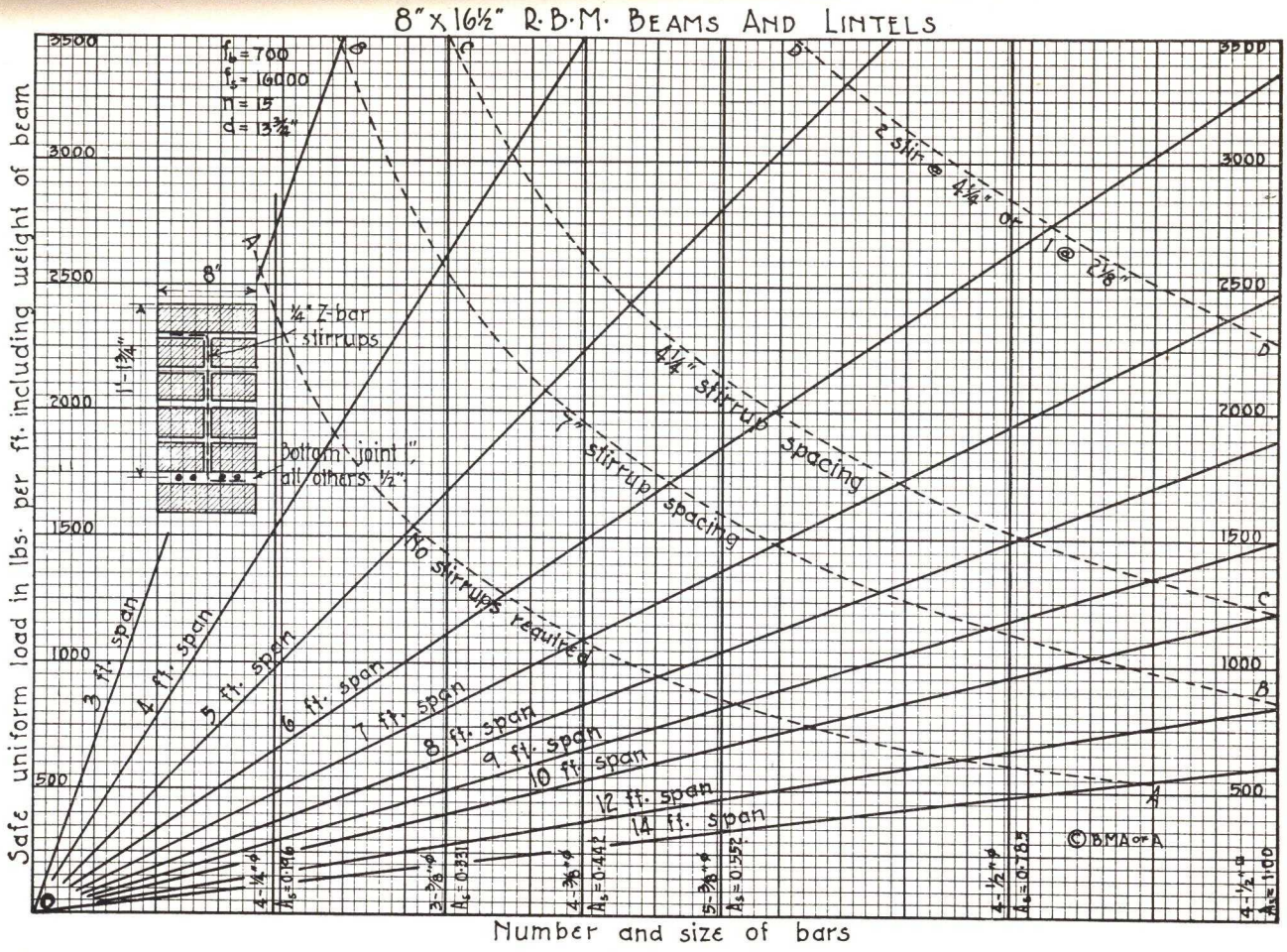

8" x 16½" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
$f_c$ = 700
$f_s$ = 16000
n = 15
d = 13 3/16"
8"
1"-1¾"
¼" Z-bar stirrups
Bottom joint 1", all others ½"
3 ft. span
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
No stirrups required
7" stirrup spacing
4¼" stirrup spacing
2 Stir. @ 4¼" or 1 @ 2⅛"
4-¼"ϕ $A_s$=0.196
3-⅜"ϕ $A_s$=0.331
4-⅜"ϕ $A_s$=0.442
5-⅜"ϕ $A_s$=0.552
4-½"ϕ $A_s$=0.785
4-½"ϕ $A_s$=1.00
© B.M.A. 9-A
3500
3000
2500
2000
1500
1000
500

*Chart No. 20*

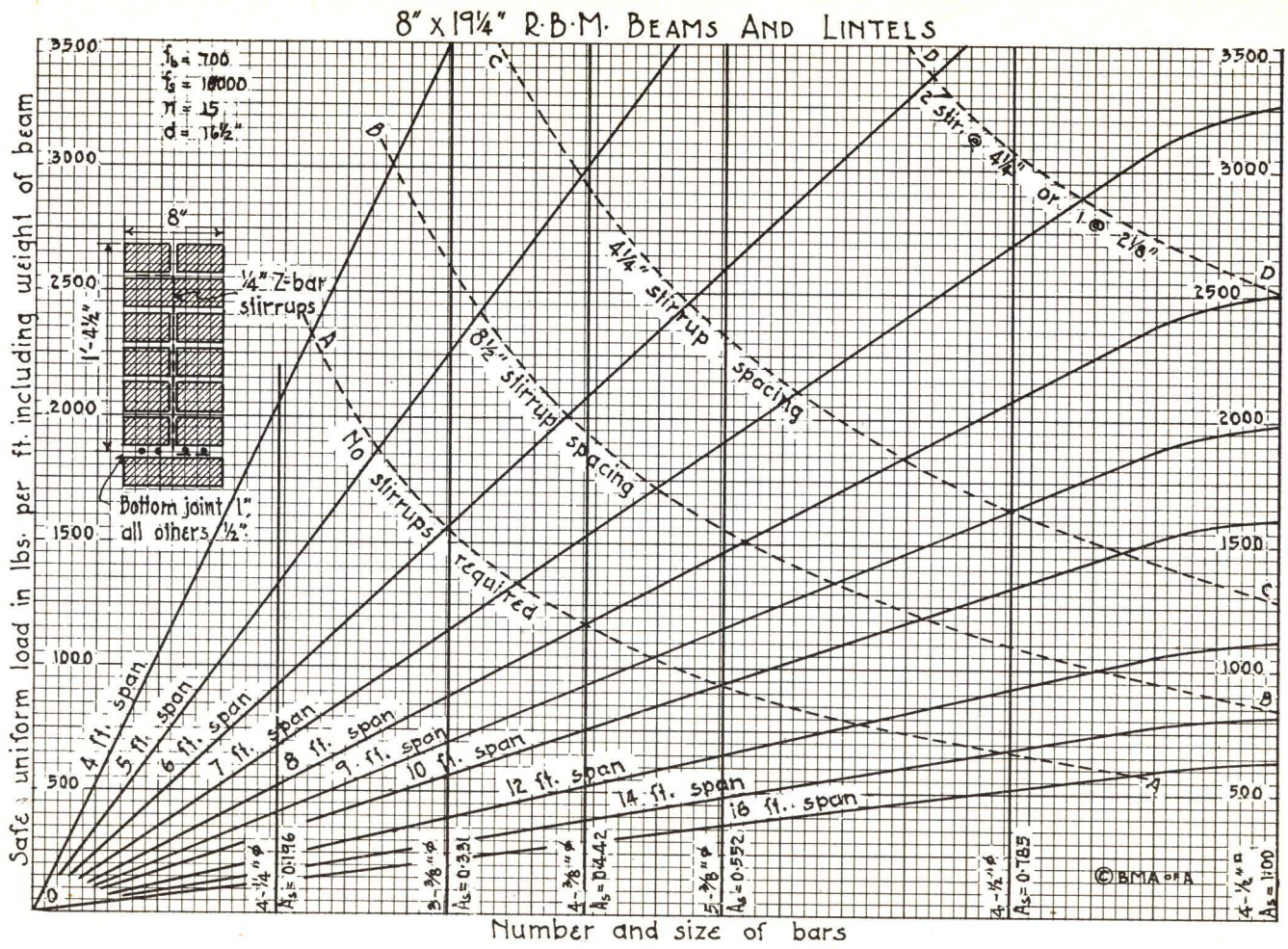

8" x 19¼" R.B.M. Beams And Lintels
$f_b$ = 700
$f_s$ = 16000
n = 15
d = 16½"
8"
1'-4½"
¼" Z-bar stirrups
Bottom joint 1", all others ½"
A
B
C
D
2 Stir. @ 4½" or 1 @ 2⅛"
4¼" stirrup spacing
8½" stirrup spacing
No stirrups required
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
3500
3000
2500
2000
1500
1000
500
0
4-¼" ø $A_s$ = 0.196
2-⅜" ø $A_s$ = 0.331
4-⅜" ø $A_s$ = 0.442
5-⅜" ø $A_s$ = 0.552
4-½" ø $A_s$ = 0.785
4-½" ø $A_s$ = 1.00
©BMA-HA
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars

*Chart No. 21*

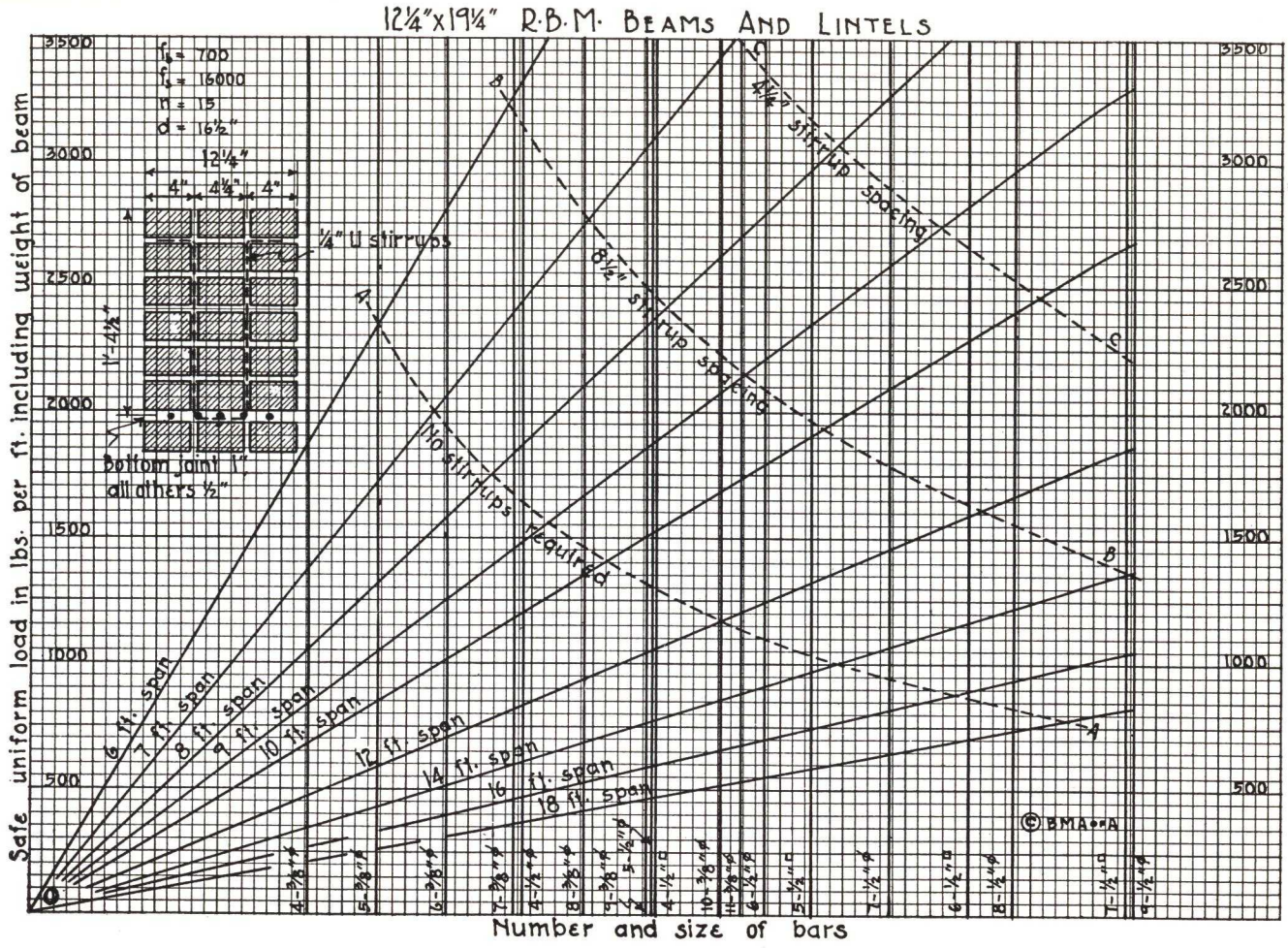

12¼"x19¼" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
f_s = 700
f_c = 16000
n = 15
d = 16½"
12¼"
4"
4½"
4"
¼" U stirrups
17-4½"
Bottom joint ½"
all others ½"
No stirrups required
4½" stirrup spacing
8½" stirrup spacing
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
A
B
C
© BMA-RA
3500
3000
2500
2000
1500
1000
500

*Chart No. 22*

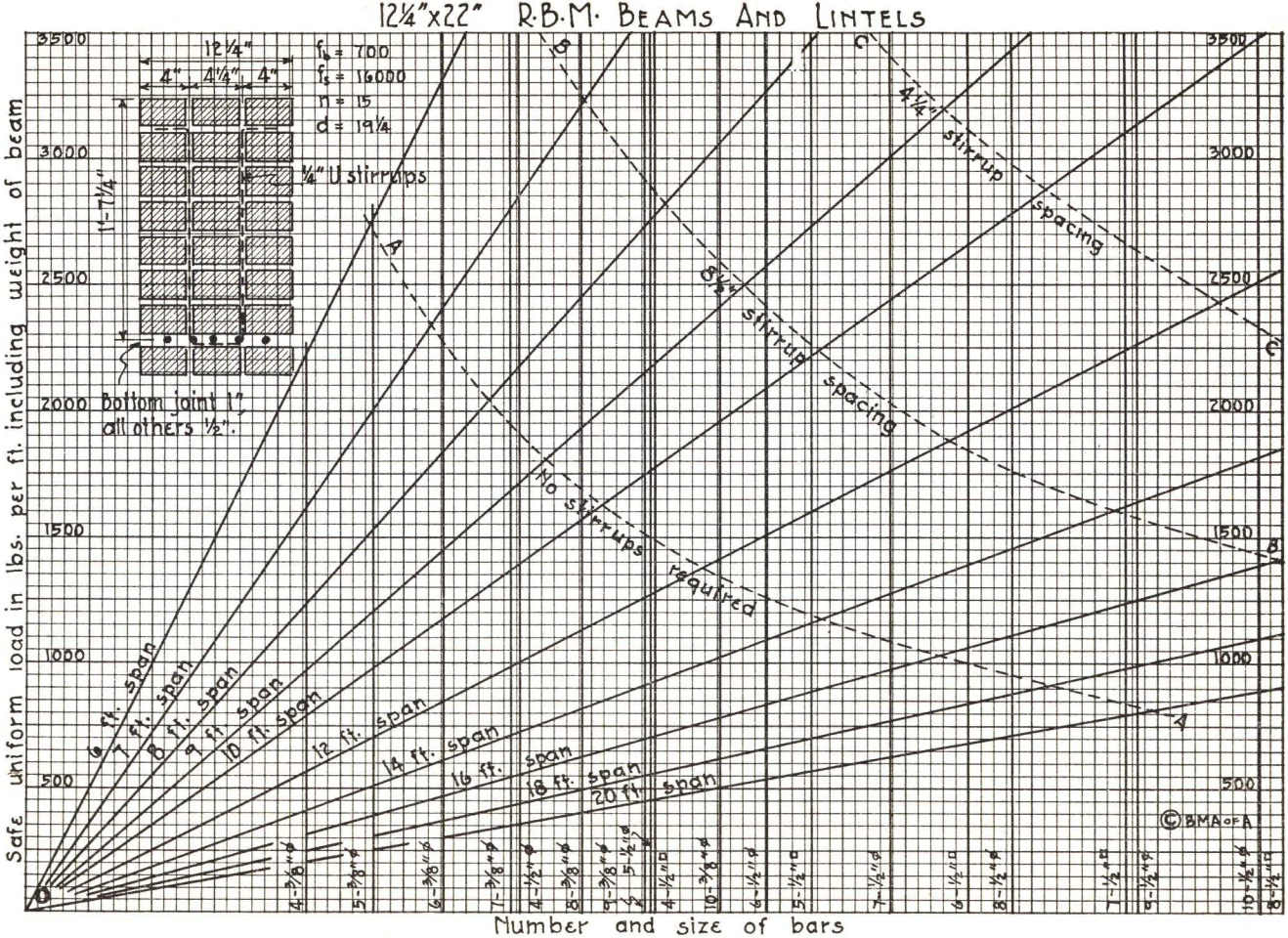

12¼"x22" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
12¼"
4" 4¼" 4"
1'-7¼"
¼" U stirrups
$f_b$ = 700
$f_s$ = 16000
n = 15
d = 19¼
Bottom joint 1"
all others ½".
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
20 ft. span
4¼" stirrup spacing
8½" stirrup spacing
No stirrups required
A
B
C
D
E
© BMA of A

*Chart No. 23*

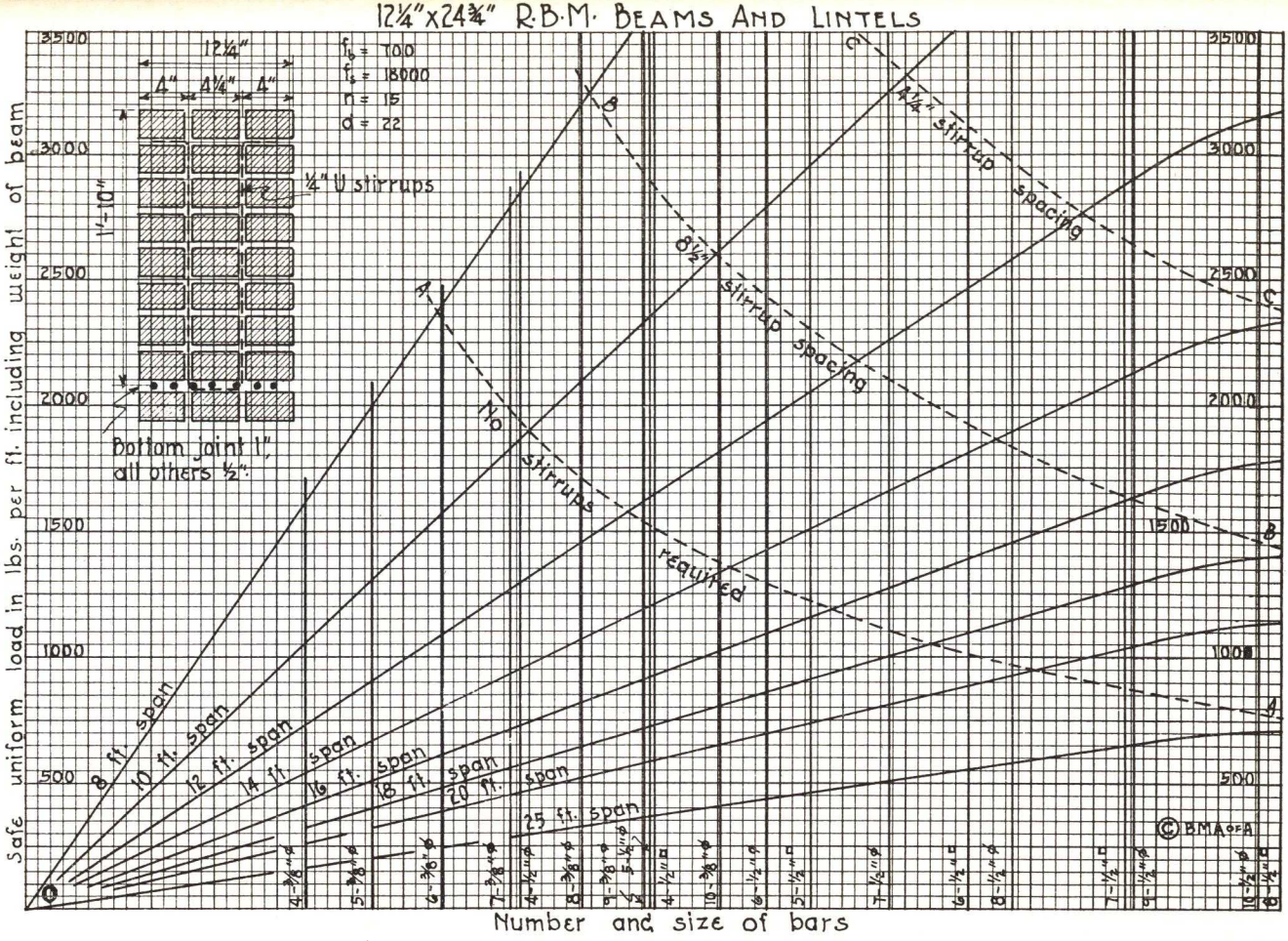

12¼"x24¾" R.B.M. Beams And Lintels
f_c = 700
f_s = 18000
n = 15
d = 22
12¼"
4"
4¼"
4"
¼" U stirrups
1'-10"
Bottom joint 1", all others ½".
Safe uniform load in lbs. per ft. including weight of beam
35000
30000
25000
20000
15000
10000
5000
8 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
20 ft. span
25 ft. span
A
B
C
No Stirrups required
8½" stirrup spacing
4¼" stirrup spacing
4-⅜"ø
5-⅜"ø
6-⅜"ø
7-⅜"ø
4-½"ø
8-⅜"ø
9-⅜"ø
5-½"ø
4-½"□
10-⅜"ø
6-½"ø
5-½"□
7-½"ø
6-½"□
8-½"ø
7-½"□
9-½"ø
10-½"ø
8-½"□
© BMA of A
Number and size of bars

*Chart No. 24*

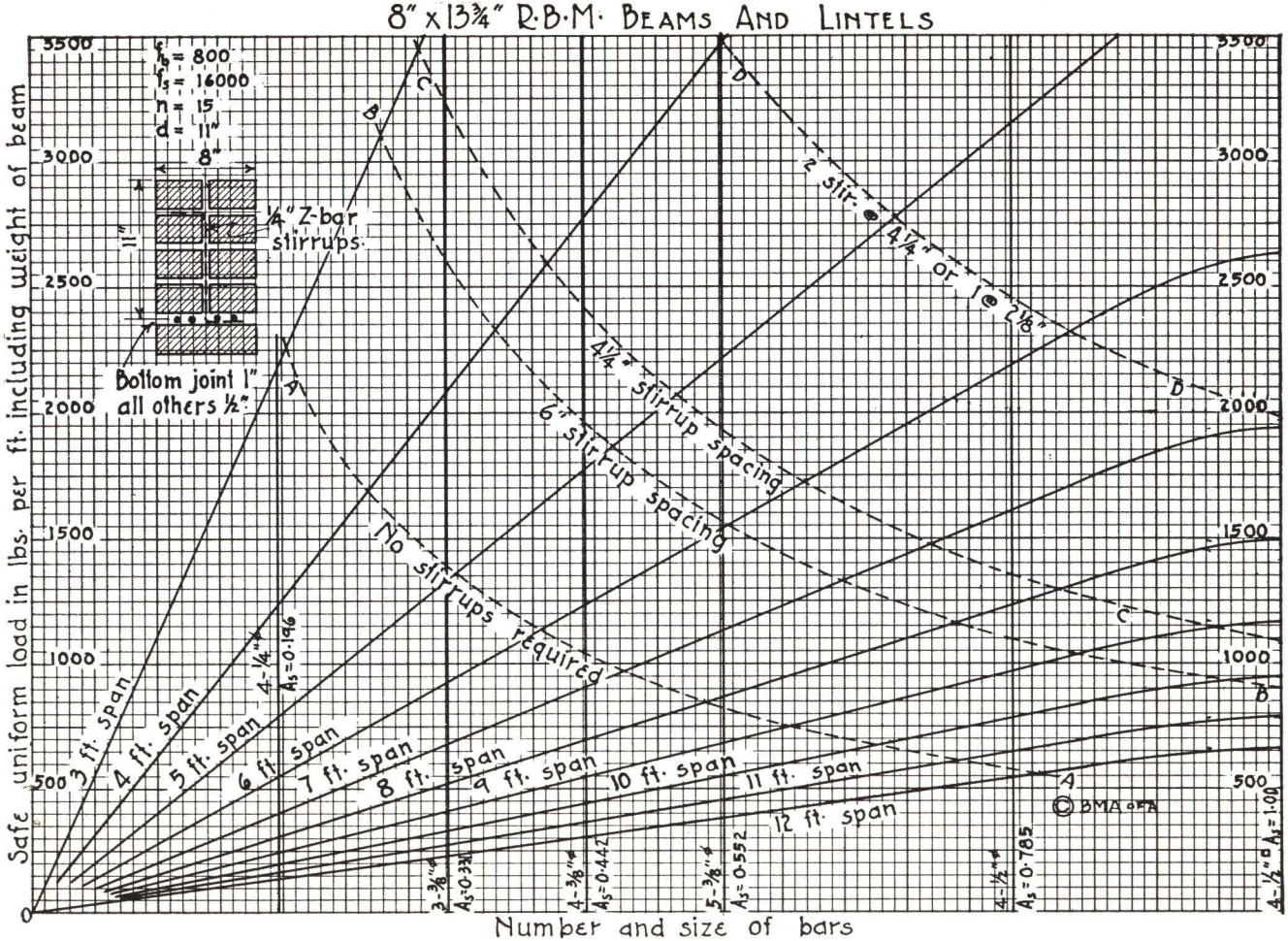

8" x 13¾" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
$f_b$ = 800
$f_s$ = 16000
n = 15
d = 11"
8"
11"
¼" Z-bar stirrups
Bottom joint 1" all others ½"
3 ft. span
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
11 ft. span
12 ft. span
4-¼" $A_s$=0.196
2-⅜" $A_s$=0.332
4-⅜" $A_s$=0.442
5-⅜" $A_s$=0.552
4-½" $A_s$=0.785
4-½" □ $A_s$=1.00
2 stir. @ 4¼" or 1 @ 2⅛"
4¼" stirrup spacing
6" stirrup spacing
No stirrups required
A
B
C
D
© BMA of A
3500
3000
2500
2000
1500
1000
500
0

*Chart No. 25*

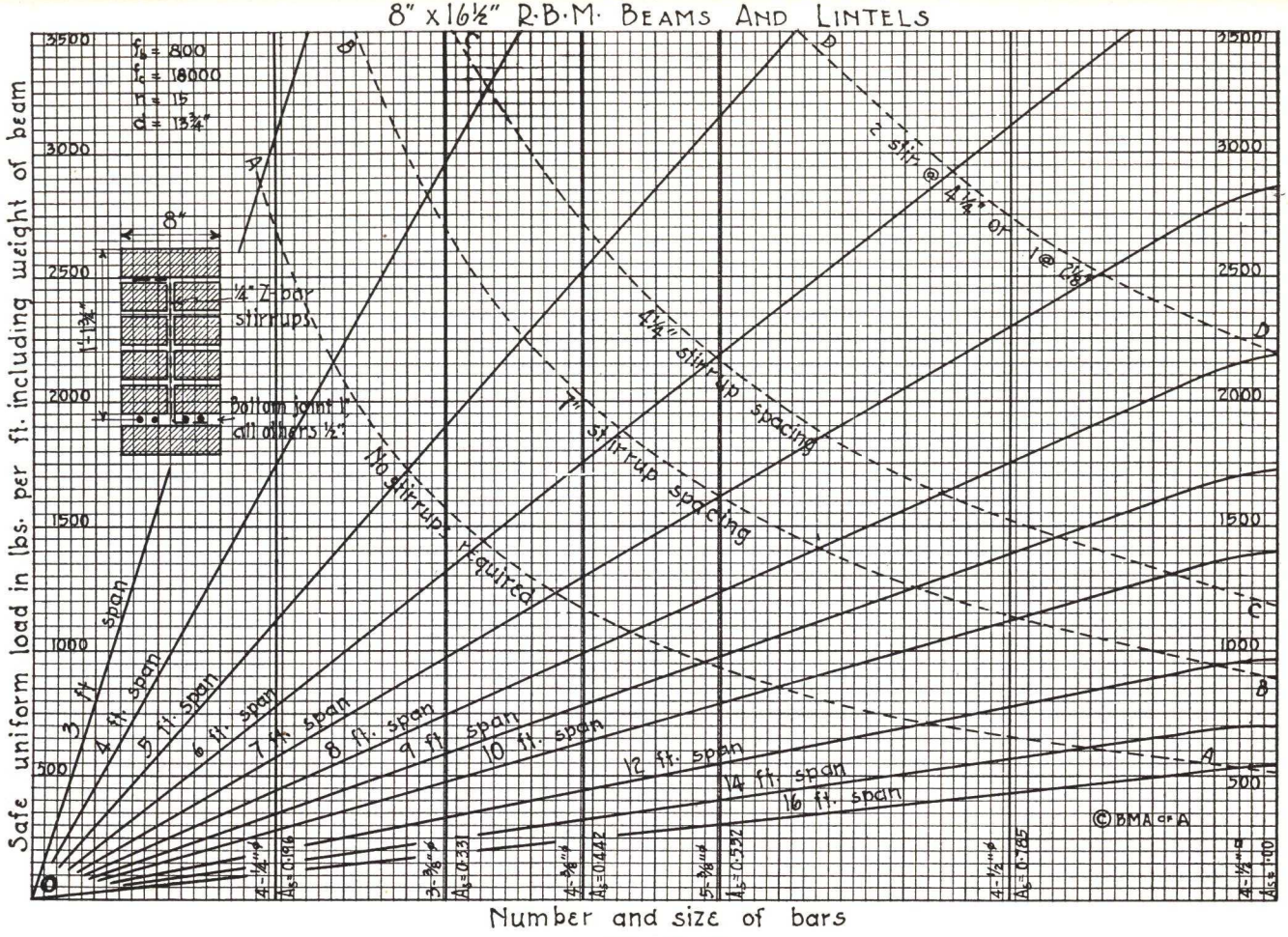

8" x 16½" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
$f_s$ = 8000
$f_c$ = 18000
n = 15
d = 13¾"
8"
16½"
¼" Z-bar stirrups
Bottom joint 1", all others ½"
No stirrup required
4¼" stirrup spacing
7" stirrup spacing
2 stirrups @ 4¼" or 1 @ 7½"
3 ft. span
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
A
B
C
D
© BMA-A

*Chart No. 26*

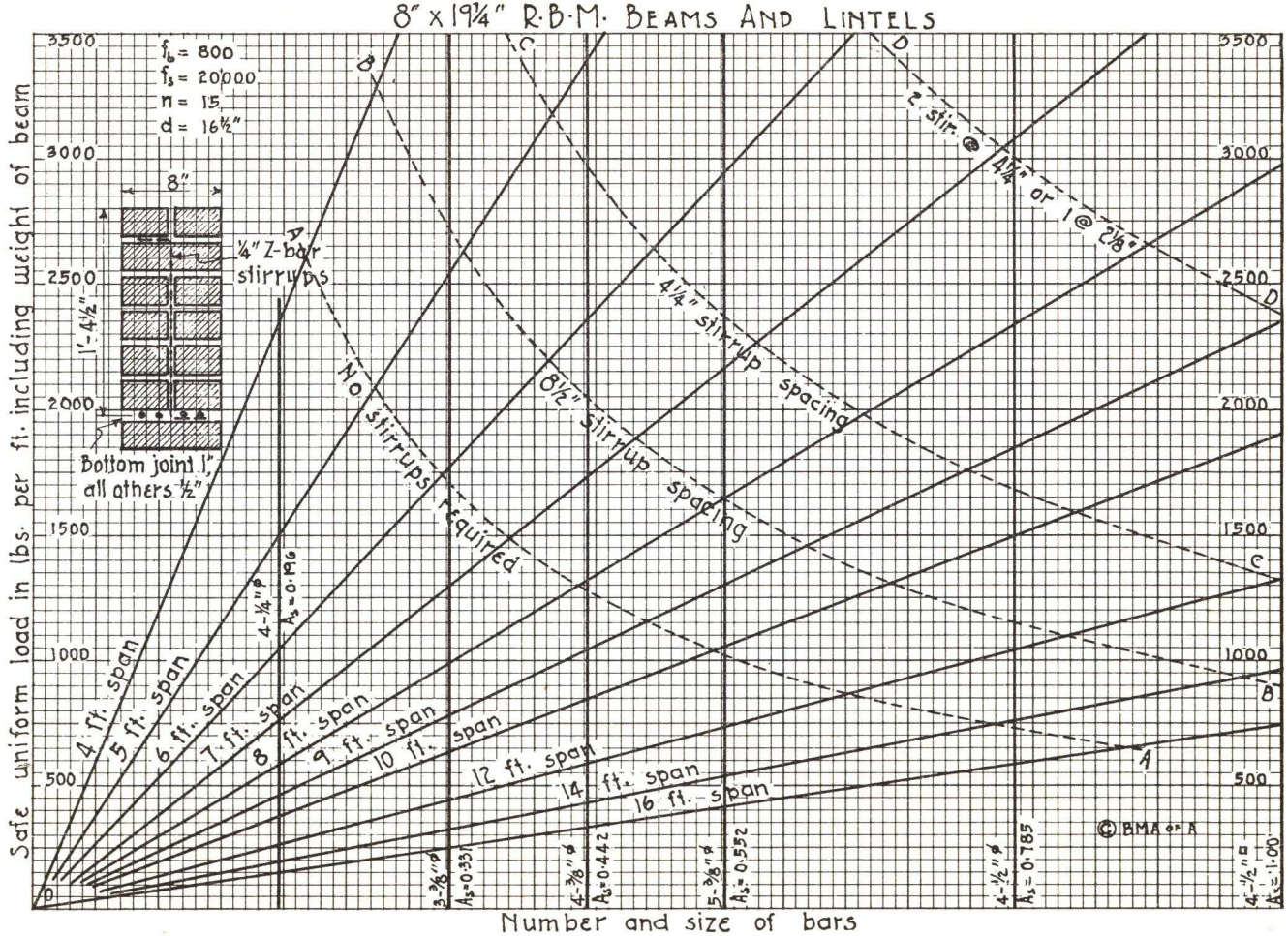

8" x 19¼" R.B.M. Beams And Lintels
$f_b$ = 800
$f_s$ = 20,000
n = 15
d = 16½"
8"
1'-4½"
¼" Z-bar stirrups
Bottom joint ½" all others ½"
Safe uniform load in lbs. per ft. including weight of beam
3500
3000
2500
2000
1500
1000
500
0
A
B
C
D
2 stir. @ 4¼" or 1 @ 2⅛"
4¼" stirrup spacing
6½" stirrup spacing
No stirrups required
4 ft. span
5 ft. span
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
4-¼" ø A$_s$=0.196
3-⅜" ø A$_s$=0.331
4-⅜" ø A$_s$=0.442
5-⅜" ø A$_s$=0.552
4-½" ø A$_s$=0.785
4-½" □ A$_s$=1.00
© BMA of A
Number and size of bars

*Chart No. 27*

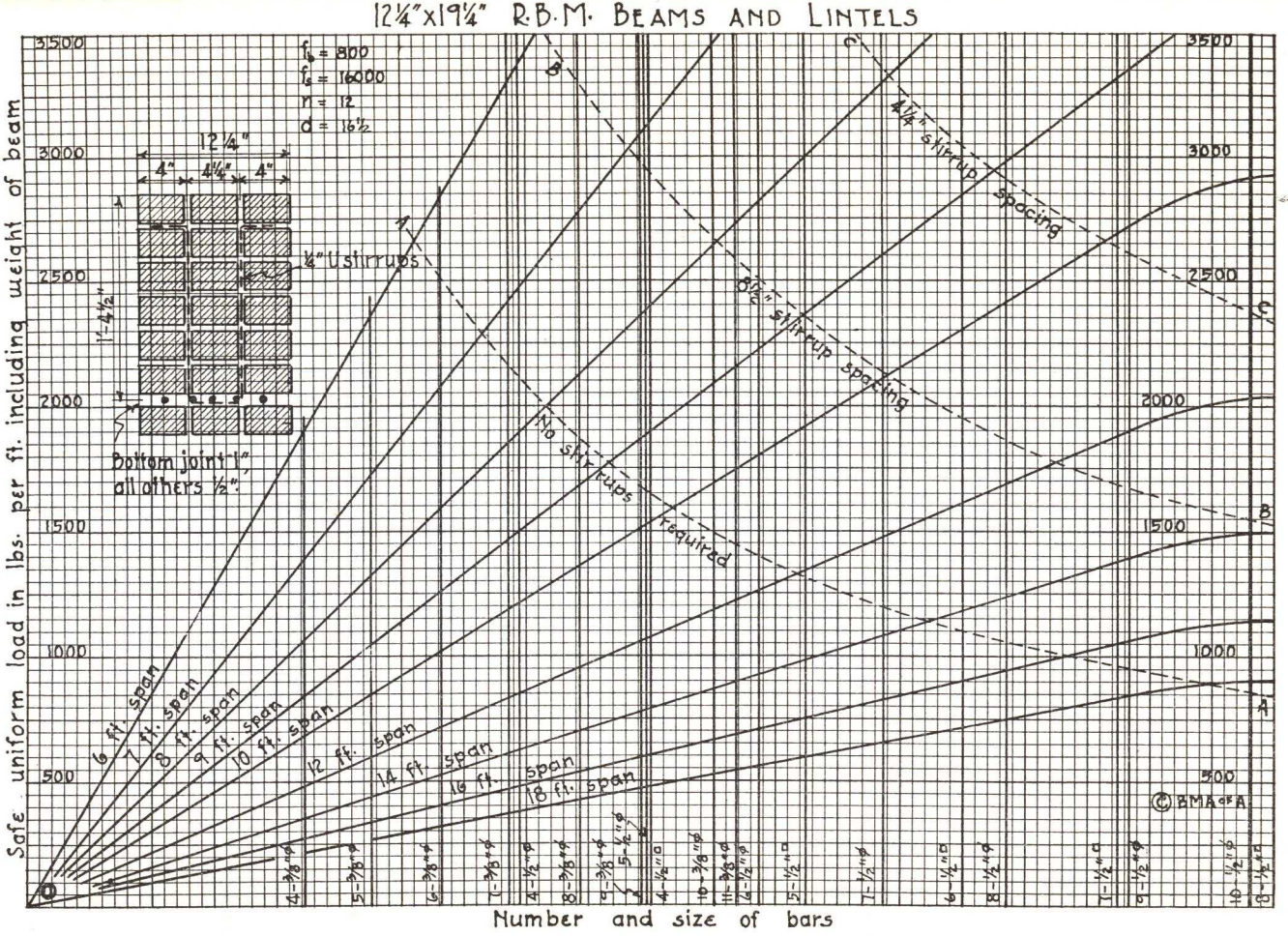

12¼"x19¼" R.B.M. BEAMS AND LINTELS
$f_s$ = 800
$f_c$ = 16000
n = 12
d = 16½
12¼"
4"
4¼"
4"
¼" U stirrups
11"-4½"
Bottom joint 1", all others ½"
4¼" stirrup spacing
8½" stirrup spacing
No stirrups required
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
A
B
C
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
© RMA of A

*Chart No. 28*

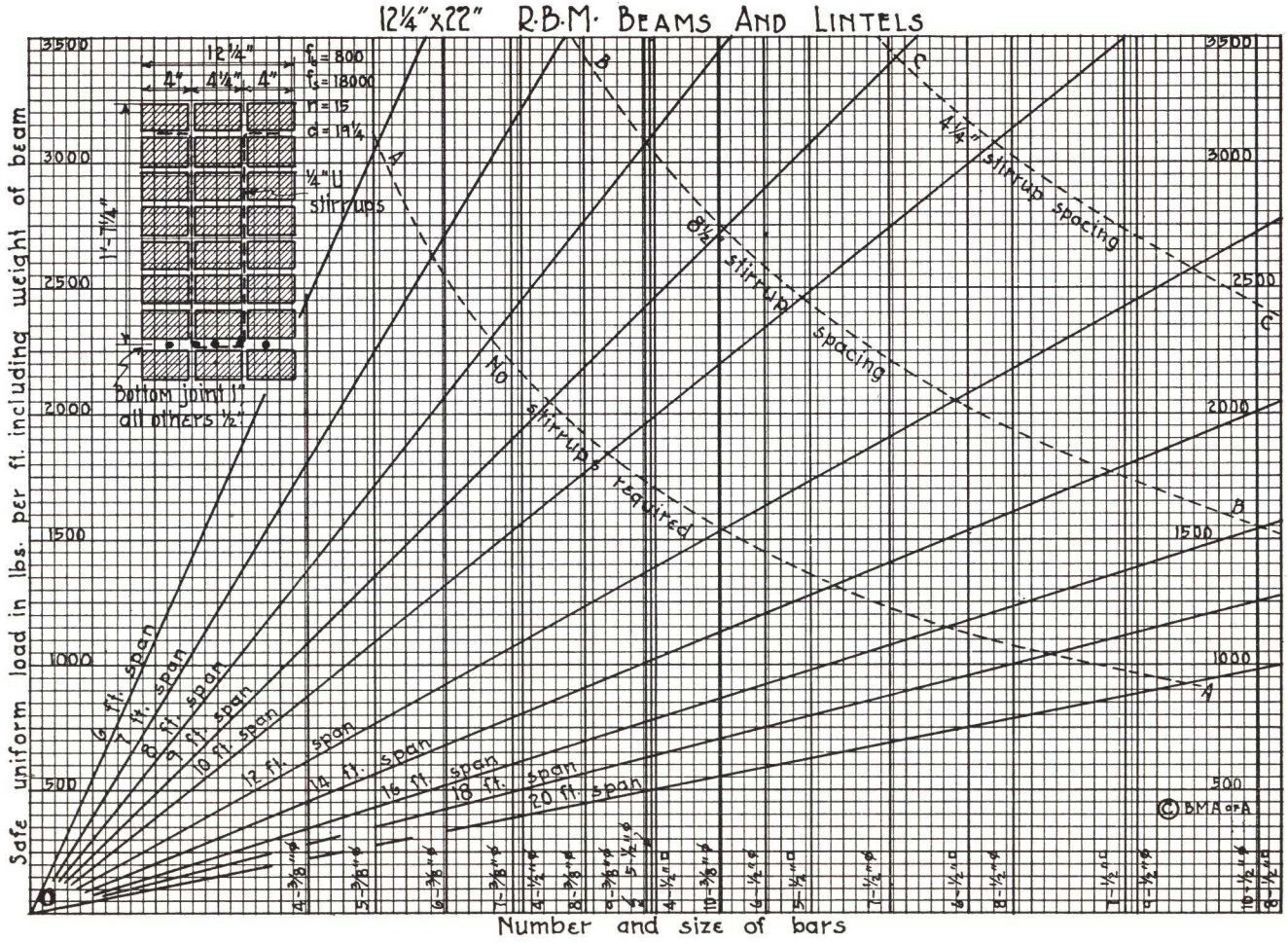

12¼"x22" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
12¼"
4" 4¼" 4"
$c_c$ = 800
$f_s$ = 18000
n = 15
d = 19¼
¼" U stirrups
1'-7¼"
Bottom joint 1"
all others ½"
No stirrups required
8¼" stirrup spacing
4¼" stirrup spacing
6 ft. span
7 ft. span
8 ft. span
9 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
20 ft. span
A
B
C
D
E
F
© BMA of A

*Chart No. 29*

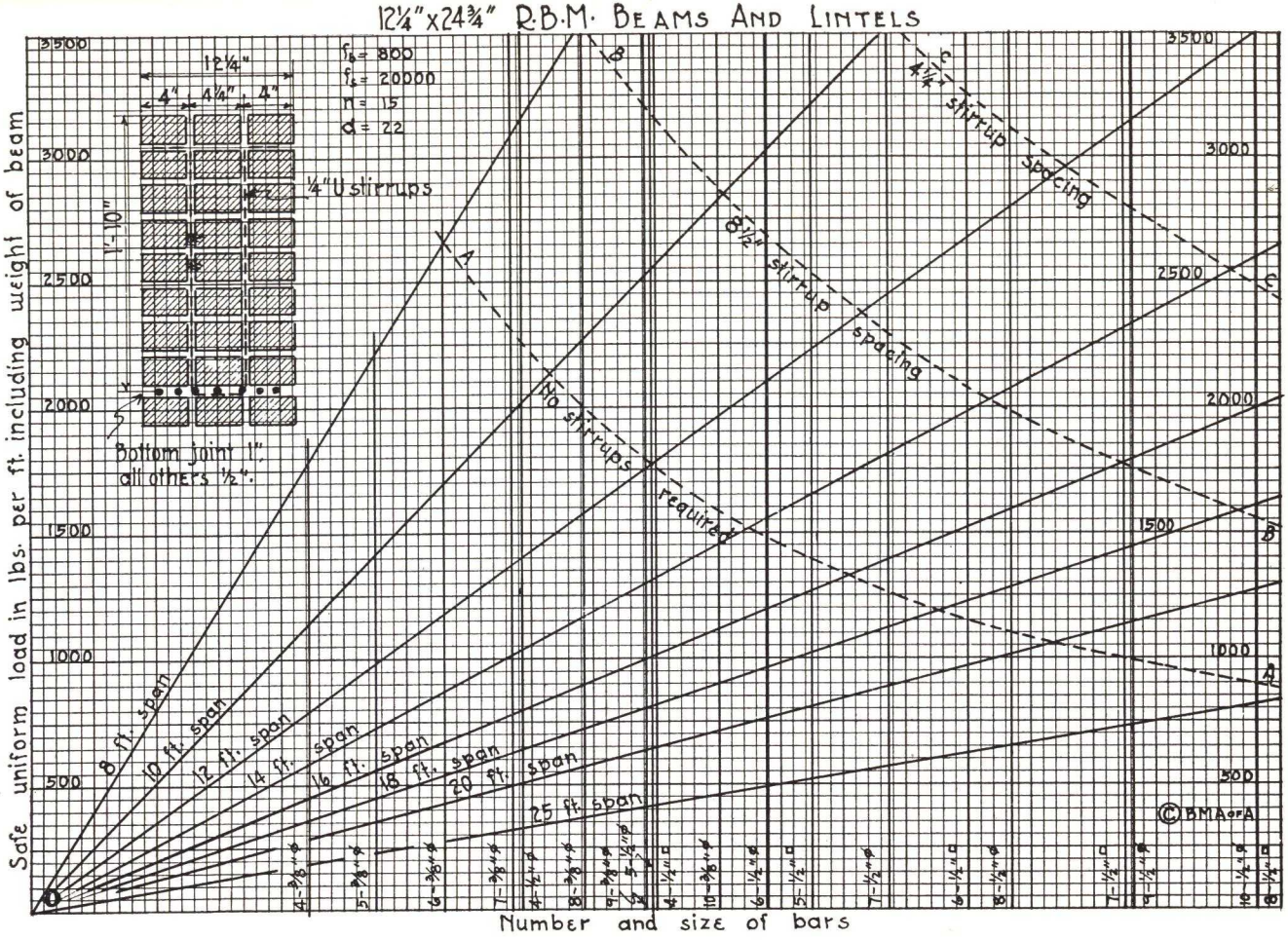

12¼"x24¾" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
12¼"
4" 4¼" 4"
1'-10"
¼" U stirrups
Bottom joint 1", all others ½".
$f_s$ = 800
$f_c$ = 20000
n = 15
d = 22
A
B
C
4¼" stirrup spacing
8½" stirrup spacing
Two stirrups required
6 ft. span
10 ft. span
12 ft. span
14 ft. span
16 ft. span
18 ft. span
20 ft. span
25 ft. span
© BMA or A

*Chart No. 30*

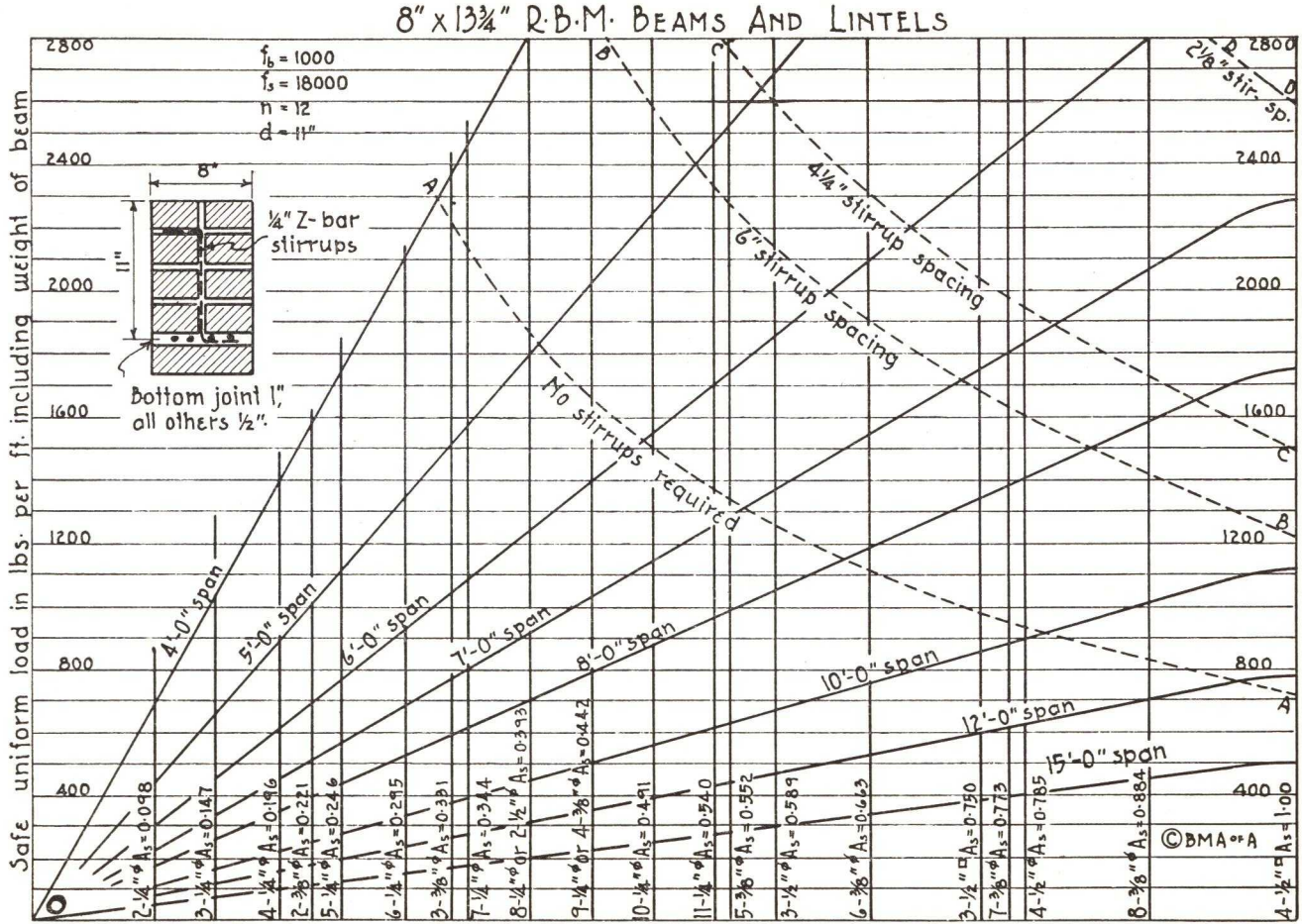

8" x 13¾" R.B.M. Beams And Lintels
$f_b$ = 1000
$f_s$ = 18000
n = 12
d = 11"
8"
11"
¼" Z-bar stirrups
Bottom joint 1", all others ½".
Safe uniform load in lbs. per ft. including weight of beam
2800
2400
2000
1600
1200
800
400
4'-0" span
5'-0" span
6'-0" span
7'-0" span
8'-0" span
10'-0" span
12'-0" span
15'-0" span
No stirrups required
6" stirrup spacing
4¼" stirrup spacing
2⅛" stir. sp.
A
B
C
D
2-¼"ϕ As=0·098
3-¼"ϕ As=0·147
4-¼"ϕ As=0·196
2-⅜"ϕ As=0·221
5-¼"ϕ As=0·246
6-¼"ϕ As=0·295
3-⅜"ϕ As=0·331
7-¼"ϕ As=0·344
8-¼"ϕ or 2-½"ϕ As=0·393
9-¼"ϕ or 4-⅜"ϕ As=0·442
10-¼"ϕ As=0·491
11-¼"ϕ As=0·540
5-⅜"ϕ As=0·552
3-½"ϕ As=0·589
6-⅜"ϕ As=0·663
3-½"□ As=0·750
7-⅜"ϕ As=0·773
4-½"ϕ As=0·785
8-⅜"ϕ As=0·884
4-½"□ As=1·00
©BMA of A
Number and size of bars

*Chart No. 31*

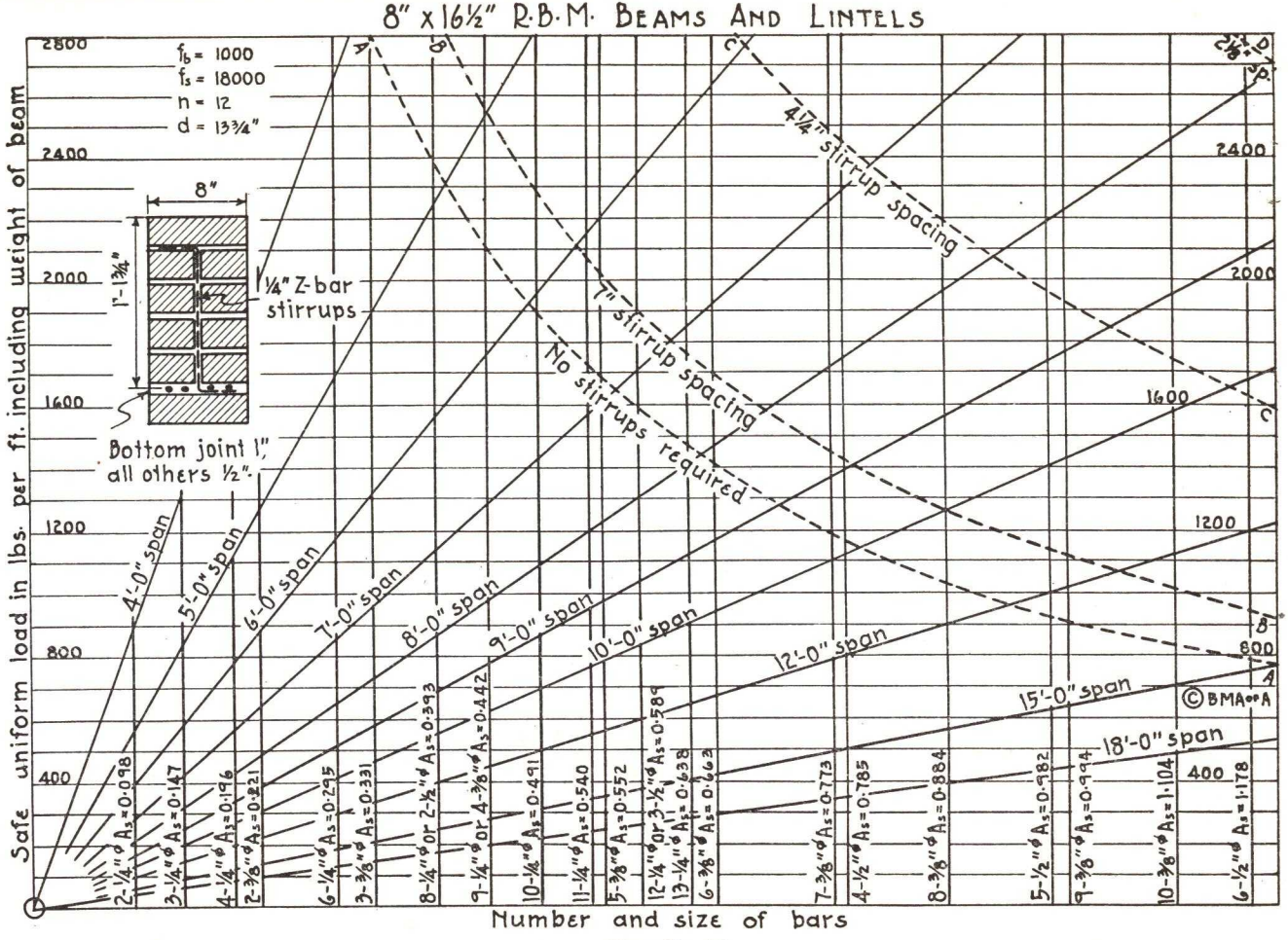

8" x 16½" R.B.M. Beams And Lintels
$f_b$ = 1000
$f_s$ = 18000
n = 12
d = 13¾"
8"
1'-1¾"
¼" Z-bar stirrups
Bottom joint ½", all others ½".
4½" stirrup spacing
7" stirrup spacing
No stirrups required
4'-0" span
5'-0" span
6'-0" span
7'-0" span
8'-0" span
9'-0" span
10'-0" span
12'-0" span
15'-0" span
18'-0" span
2-¼" ϕ As=0.098
3-¼" ϕ As=0.147
4-¼" ϕ As=0.196
2-⅜" ϕ As=0.221
6-¼" ϕ As=0.295
3-⅜" ϕ As=0.331
8-¼" ϕ or 2-½" ϕ As=0.393
9-¼" ϕ or 4-⅜" ϕ As=0.442
10-¼" ϕ As=0.491
11-¼" ϕ As=0.540
5-⅜" ϕ As=0.552
12-¼" ϕ or 3-½" ϕ As=0.589
13-¼" ϕ As=0.638
6-⅜" ϕ As=0.662
7-⅜" ϕ As=0.773
4-½" ϕ As=0.785
8-⅜" ϕ As=0.884
5-½" ϕ As=0.982
9-⅜" ϕ As=0.994
10-⅜" ϕ As=1.104
6-½" ϕ As=1.178
2800
2400
2000
1600
1200
800
400
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
© BMA-A

*Chart No. 32*

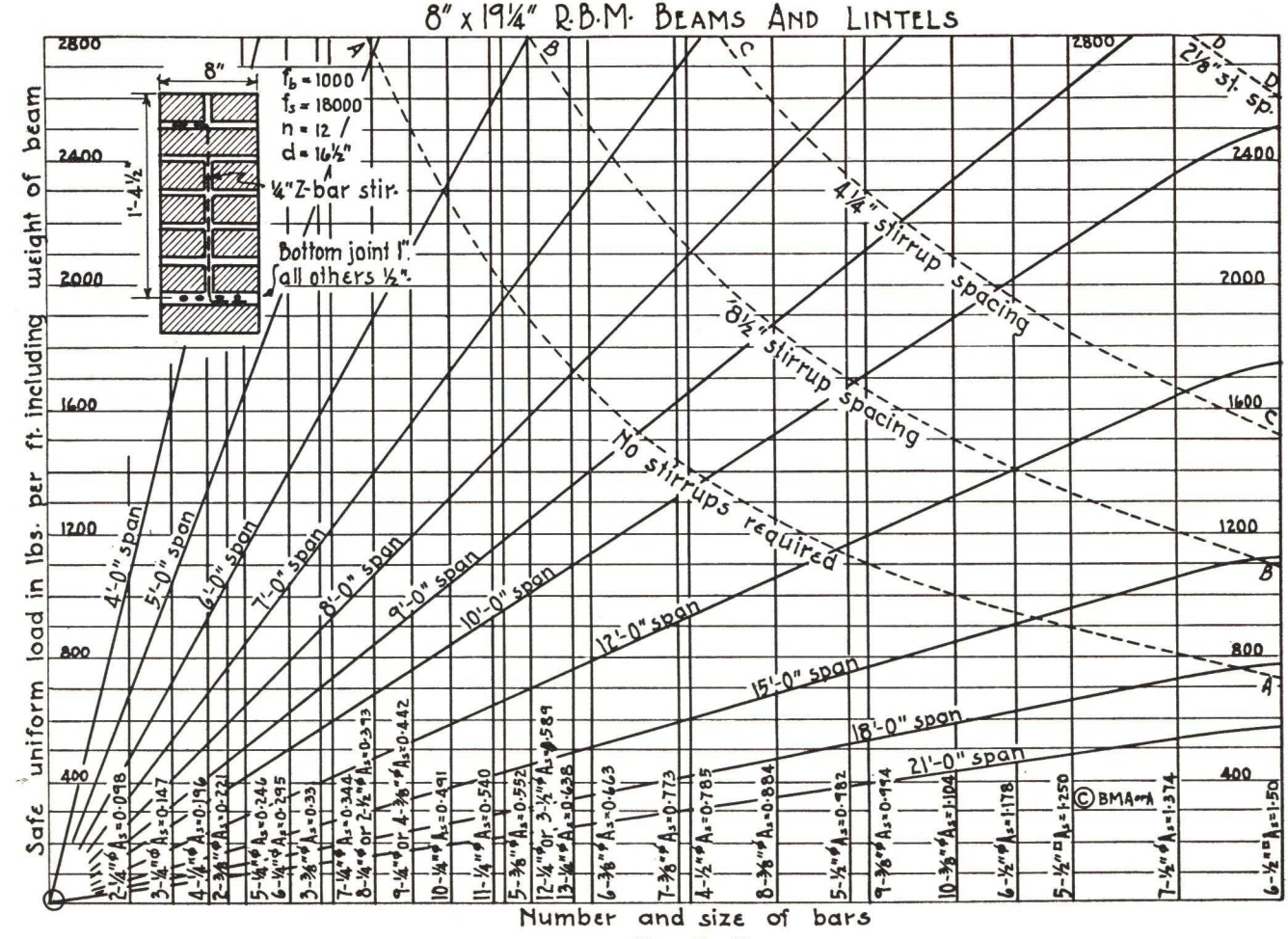

8" x 19¼" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
8"
1'-4½"
$f_b$ = 1000
$f_s$ = 18000
n = 12
d = 16½"
¼" Z-bar stir.
Bottom joint ½".
all others ½".
4'-0" span
5'-0" span
6'-0" span
7'-0" span
8'-0" span
9'-0" span
10'-0" span
12'-0" span
15'-0" span
18'-0" span
21'-0" span
4¼" stirrup spacing
8½" stirrup spacing
No stirrups required
2⅛" st. sp.
2-¼"ø As=0.098
3-¼"ø As=0.147
4-¼"ø As=0.196
2-⅜"ø As=0.221
5-¼"ø As=0.245
6-¼"ø As=0.295
3-⅜"ø As=0.33
7-¼"ø As=0.344
8-¼"ø or 2-½"ø As=0.393
9-¼"ø or 4-⅜"ø As=0.442
10-¼"ø As=0.491
11-¼"ø As=0.540
5-⅜"ø As=0.552
12-¼"ø or 3-½"ø As=0.589
13-¼"ø As=0.638
6-⅜"ø As=0.663
7-⅜"ø As=0.773
4-½"ø As=0.785
8-⅜"ø As=0.884
5-½"ø As=0.982
9-⅜"ø As=0.994
10-⅜"ø As=1.104
6-½"ø As=1.178
5-½"□ As=1.250
7-½"ø As=1.374
6-½"□ As=1.50
© BMA

*Chart No. 33*

12¼"x19¼" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
$f_c$ = 1000 /
$f_s$ = 18000
n = 12
d = 16½"
¼" U stirrups
Bottom joint 1"
all others ½"
8½" stirrup spacing
No stirrups required
6'-0" span
7'-0" span
8'-0" span
9'-0" span
10'-0" span
11'-0" span
12'-0" span
14'-0" span
17'-0" span
21'-0" span
25'-0" span
©BMA°A
2800
2400
2000
1600
1200
800
400
4-¼"ø As=0.196
2-⅜"ø As=0.221
6-¼"ø As=0.295
3-⅜"ø As=0.331
8-¼"ø As=0.393
4-⅜"ø As=0.442
10-¼"ø As=0.491
5-⅜"ø As=0.552
12-¼"ø or 5-½"ø As=0.589
6-⅜"ø As=0.663
15-¼"ø As=0.736
7-⅜"ø As=0.773
17-¼"ø As=0.834
8-⅜"ø As=0.884
19-¼"ø As=0.933
5-½"ø As=0.982
22-¼"ø As=1.08
24-¼"ø or 6-½"ø As=1.178
11-⅜"ø As=1.214
12-⅜"ø As=1.325
7-½"ø As=1.374
13-⅜"ø As=1.436
6-½"□ As=1.502
8-½"ø As=1.57
15-⅜"ø As=1.656
16-⅜"ø or 9-½"ø As=1.767
17-⅜"ø As=1.877
10-½"ø As=1.963
8-½"□ As=2.00

*Chart No. 34*

12¼" x 22" R.B.M. Beams and Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
Chart No. 35
f_c = 1000 V
f_s = 20000
n = 12
d = 19¼"
¼" U stirrups
Bottom joint 1"; all others ½".
12¼"
4" 4¼" 4"
1'-7¼"
No stirrups required
8½" stirrup spacing
6'-0" span
7'-0" span
8'-0" span
10'-0" span
12'-0" span
15'-0" span
18'-0" span
21'-0" span
25'-0" span
2-½"ø As=0.098
4-¼"ø As=0.196
2-⅜"ø As=0.221
6-¼"ø As=0.295
3-⅜"ø As=0.331
8-¼"ø or 2-½"ø As=0.393
4-⅜"ø As=0.442
10-¼"ø As=0.491
5-⅜"ø As=0.552
12-¼"ø As=0.589
6-⅜"ø As=0.663
15-¼"ø As=0.736
7-⅜"ø As=0.773
17-¼"ø As=0.834
8-⅜"ø As=0.884
19-¼"ø As=0.933
5-½"ø As=0.982
22-¼"ø As=1.08
24-¼"ø or 6-½"ø As=1.178
11-⅜"ø As=1.214
12-⅜"ø As=1.325
7-½"ø As=1.374
13-⅜"ø As=1.436
6-½"□ As=1.50
8-½"ø As=1.57
15-⅜"ø As=1.656
9-½"ø As=1.767
17-⅜"ø As=1.877
10-½"ø As=1.963
8-½"□ As=2.00
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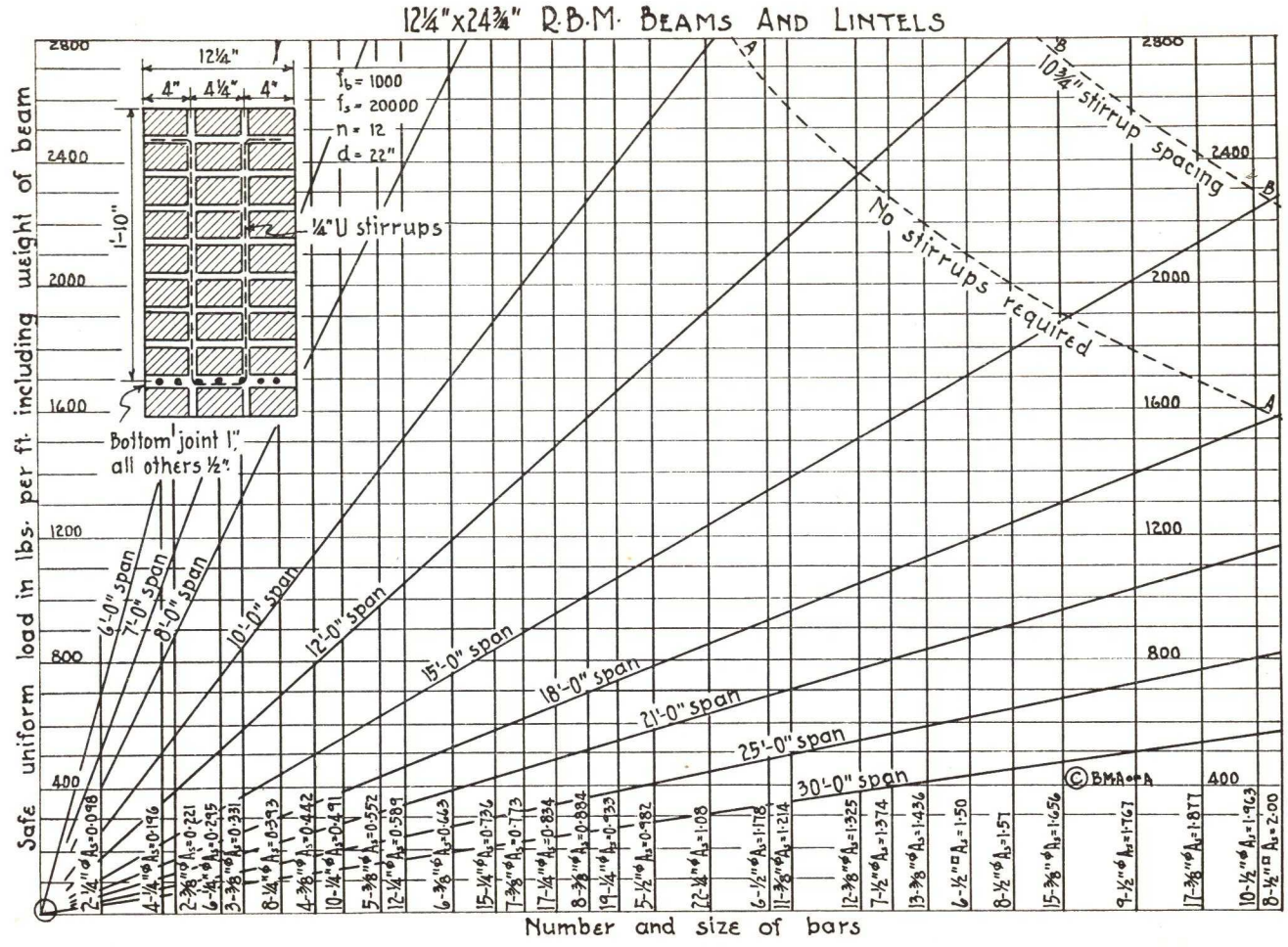
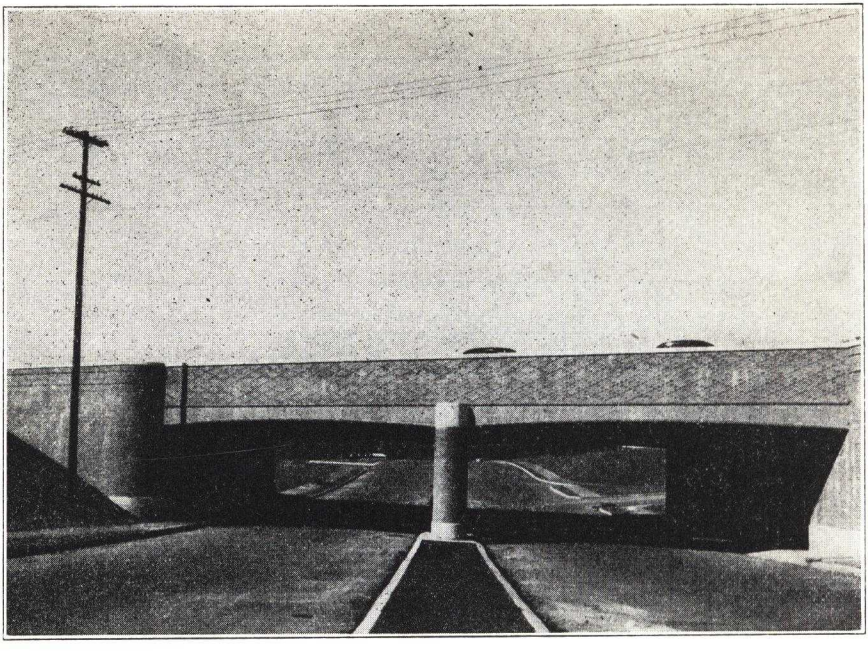
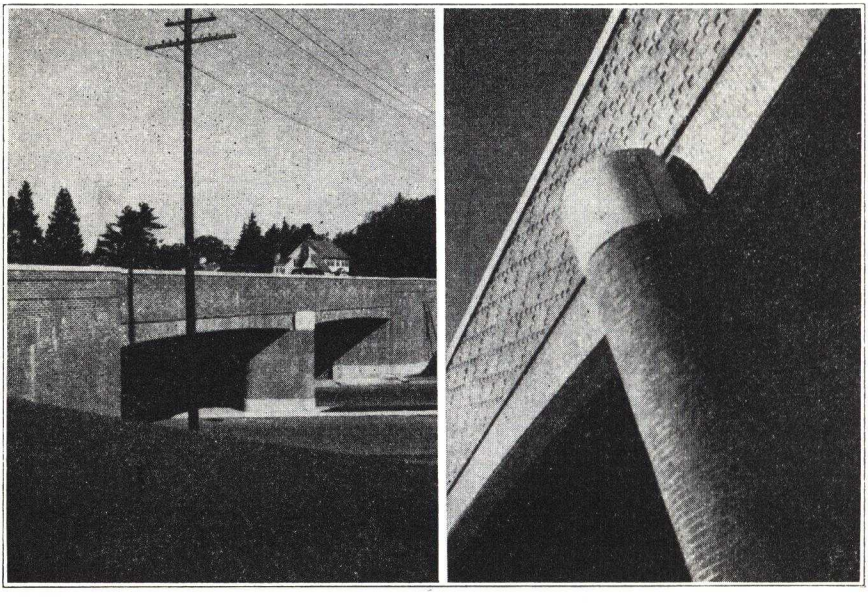

12¼"x24¾" R.B.M. Beams And Lintels
Safe uniform load in lbs. per ft. including weight of beam
Number and size of bars
12¼"
4"
4¼"
4"
1'-10"
$f_c$ = 1000
$f_s$ = 20000
n = 12
d = 22"
¼" U stirrups
Bottom joint 1", all others ½"
No stirrups required
10¾" stirrup spacing
6'-0" span
7'-0" span
8'-0" span
10'-0" span
12'-0" span
15'-0" span
18'-0" span
21'-0" span
25'-0" span
30'-0" span
2800
2400
2000
1600
1200
800
400
2-¼" ϕ $A_s$=0.098
4-¼" ϕ $A_s$=0.196
2-⅜" ϕ $A_s$=0.221
6-¼" ϕ $A_s$=0.295
3-⅜" ϕ $A_s$=0.331
8-¼" ϕ $A_s$=0.393
4-⅜" ϕ $A_s$=0.442
10-¼" ϕ $A_s$=0.491
5-⅜" ϕ $A_s$=0.552
12-¼" ϕ $A_s$=0.589
6-⅜" ϕ $A_s$=0.663
15-¼" ϕ $A_s$=0.736
7-⅜" ϕ $A_s$=0.773
17-¼" ϕ $A_s$=0.834
8-⅜" ϕ $A_s$=0.884
19-¼" ϕ $A_s$=0.933
5-½" ϕ $A_s$=0.982
22-¼" ϕ $A_s$=1.08
6-½" ϕ $A_s$=1.178
11-⅜" ϕ $A_s$=1.214
12-⅜" ϕ $A_s$=1.325
7-½" ϕ $A_s$=1.374
13-⅜" ϕ $A_s$=1.436
6-½" □ $A_s$=1.50
8-½" ϕ $A_s$=1.57
15-⅜" ϕ $A_s$=1.656
© BMA - A
9-½" ϕ $A_s$=1.767
17-⅜" ϕ $A_s$=1.877
10-½" ϕ $A_s$=1.963
8-½" □ $A_s$=2.00

*Chart No. 36*





*Illustrating the architectural effects and possibilities of reinforced brick masonry in bridges, piers and retaining walls*

# CHAPTER 11

# R-B-M COLUMNS AND PIERS

## 1101. PIERS

A pier is usually considered to be a vertical compression member in which the unsupported length is not greater than six times its least lateral dimension. If a pier sustains only vertical loads whose resultant coincides with the axis of the pier, the unit compressive stress is uniformly distributed over the base. The minimum area of such a pier is determined by dividing the total load by the allowable unit compressive stress in the masonry. Practical considerations, however, usually necessitate a larger section.

## 1102. ECCENTRIC LOADING

An eccentric load is one so placed that its line of action does not coincide with the gravity axis of the pier or column. An eccentric load increases the unit compressive stress on the side of the eccentricity and decreases the unit compressive stress on the side opposite. If the eccentricity is large enough, the extreme fiber on the far side of the pier or column might be in tension. The standard formula for the extreme fiber stress due to eccentric loads upon solid rectangular columns or piers of homogeneous materials is as follows:

$$F_b = \frac{P}{A}\left(1 \pm \frac{6e}{d}\right) \quad \ldots\ldots (15)$$

where $F_b$ = total unit stress at outside edge of pier and column
P = total load applied
A = cross-sectional area of pier or column
e = the eccentricity of the load
d = the total lateral dimension in the direction of the eccentricity.

The equation with the plus sign gives the maximum stress at the edge near the eccentric loading, while the minus sign is used to obtain the stress in the extreme fiber on the other side. When the result is positive, the stress is compression; when negative, it is tension. The negative result indicates that the eccentricity of the loading has caused the resultant to fall outside the kern; in which event, the pier should be redesigned or vertical reinforcing steel placed in the tension side of the pier, extending into and anchoring in footing.

The sectional area of the pier is usually increased to provide for the additional stress caused by eccentric loads. The resultant of all loads on a pier should fall within the kern, which in a rectangular pier is a parallelogram in the center of the pier base and whose diagonals are the middle third of the axes of said base.

Eccentric loading is frequently caused by placing bearing plates of beams or trusses close to the inside edge of the pier. The deflection of beams results in an inclined reaction which causes bending stresses in piers and columns. From a practical standpoint, therefore, it is advisable to place some vertical reinforcement near that surface of any pier in which tensile stresses are likely to develop.

## 1103. COLUMNS

A column is a vertical structural member whose length exceeds six times its least lateral dimension and differs from other compression members in that a column is usually designed to resist bending, which is assumed to take place. A short column is one whose unsupported length does not exceed ten times its least lateral dimension, while the long column has a ratio of length to least width in excess of ten.

An R-B-M column may be reinforced by means of:

(a) structural steel shapes, sufficiently rigid to have strength as a column;
(b) longitudinal or vertical rods;
(c) hoops or bands; or
(d) a combination of (a) and (c), (b) and (c) or all three.

## 1104. COLUMNS REINFORCED WITH STRUCTURAL STEEL SHAPES

A combination column, in which the steel forms a column in itself and is simply fireproofed with brick masonry, cannot properly be called an R-B-M column. There are, however, combinations of brick masonry and structural steel in which the steel may be designed to take the greater part of the load, while the brick masonry is so constructed as to take some of the load and also exert a supporting effect in addition to its fireproofing qualities. In these columns, the brick masonry restrains the steel from later deflection and actually takes considerable load along with the steel, thereby increasing the carrying capacity of the column. The tests at the Bureau of Standards, discussed in Section 1105, indicated that the ultimate compressive strength of the structural steel columns was increased 75% when constructed in combination with the brick masonry.

## 1105. TESTS ON STEEL COLUMNS INCASED IN BRICK MASONRY

The National Bureau of Standards made some tests on structural steel columns incased in brick masonry reported in Bureau of Standards Journal of Research, Vol. 10, RP520, January 1933. The following data and comments have been taken from that paper.

The purpose of the tests was to determine for design purposes, whether the strength of steel columns incased in brick walls should be calculated by considering them as long, bare columns which would fail by bending at low loads, or whether the brick masonry could be relied upon to restrain the lateral bending and enable the long steel columns to carry higher loads similar to short columns of the same material.

Nine 6″ H-shaped steel columns, weighing 20 lbs. per ft. and 23 ft. long were tested in compression. Three of the steel columns were tested without any incasement. Six of them were incased in brick walls 14 in. thick, 6 feet long, carried up to within approximately 8 in. of the top of the steel. The test load was applied to the top of the steel section.

The brick masonry was composed of clay brick, having the following average properties: Compressive strength, 5253 lbs. per sq. inch; modulus of rupture, 800 lbs. per sq. inch; 5-hr. boiling absorption, 11.8%. The mortar was composed of one part portland cement to three parts of damp sand by volume, with hydrated lime added equal to 10% of the cement by weight.

Three of the incased columns had the H-section oriented in one direction, while the other three had the H-section oriented perpendicular to that of the first three. It was found, however, that the orientation of the steel column section with respect to the face of the brick wall had no effect upon the strength of the incased columns.

The incased columns were much stronger than the bare specimens. According to Messrs. A. L. Harris, A. H. Stang and J. W. McBurney, who performed the tests, this increase in strength may arise from two different causes. The load may be partially transferred from the steel to the brick incasement by the bond between them. Added

strength from this cause might be expected in columns of any length. Long columns, when laterally unrestrained, fail by bending at loads much smaller than the yield strength of the steel. Incasement of long columns would act as a continuous lateral restraint which might be sufficient to prevent lateral bending and enable the columns to support greater loads.

All of the incased columns failed by yielding, followed by local buckling of the short unincased portion of the steel column above the brickwork. The only failure in the brickwork was at the top, where fine cracks appeared between the brick wythes, apparently caused by the local buckling of the unincased part of the steel column.

**TABLE NO. 40**

**RESULTS OF COMPRESSIVE TESTS OF THE COLUMNS AND OF AUXILIARY SHORT LENGTHS**

| Column | Condition | Computed area of steel column | Unincased length of steel column & of the short length | Height of Wall | Time between completion of walls and test | Yield point of steel samples | Compressive Strength | | Load carried by | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Column | Short Length | Steel Column | Brick Wall |
| | | Sq. In. | In. | Ft.-In. | Days | Lb./sq. in. | Lb./sq. in. | Lb./sq. in. | % | % |
| 1 | Bare | 5.90 | | | | 42,800 | 23,900 | | 100 | 0 |
| 2 | Bare | 5.90 | | | | 47,500 | 23,000 | | 100 | 0 |
| 3 | Bare | 5.85 | | | | 42,000 | 23,100 | | 100 | 0 |
| Average | | | | | | 44,100 | 23,300 | | 100 | 0 |
| A[1] | Incased | 5.85 | 7½ | 22–4½ | 28 | 41,100 | 40,000 | 41,200 | 18 | 82 |
| A[2] | Incased | 6.02 | 7½ | 22–4½ | 31 | 39,000 | 41,500 | 43,200 | 17 | 83 |
| A[3] | Incased | 5.98 | 11 | 22–1 | 30 | 40,100 | 40,700 | 38,300 | 19 | 81 |
| Average | | | | | 30 | 40,100 | 40,700 | 40,900 | 18 | 82 |
| B[1] | Incased | 6.03 | 10 | 22–2 | 29 | 39,000 | 38,800 | 40,600 | 16 | 84 |
| B[2] | Incased | 6.14 | 8 | 22–4 | 31 | 41,400 | 42,100 | 41,500 | 19 | 81 |
| B[3] | Incased | 6.10 | 8½ | 22–3½ | 31 | 40,500 | 41,100 | 42,200 | 16 | 84 |
| Average | | | | | 30 | 40,300 | 40,700 | 41,400 | 17 | 83 |

The brick incased columns were removed from the testing machine by an overhead crane with a chain slung around the wall a few feet above the middle. The walls were laid horizontally on the floor without a brick falling out. The fact that these walls could be handled in this manner without any indication of failure in the brick walls is evidence that the brick walls were not appreciably affected by the test loads. See page 257.

Compressometer readings over the 150-in. gauge length, centered at mid-height, showed that the steel in the incased columns was stressed, on the average, less than 20% as much as was the steel in the unincased columns at the same loading. The lateral deflection of the incased columns was very small. In no case did it exceed 0.03 inch under the maximum load.

From the fact that the steel in the incased columns carried only a small proportion of the load, it is evident that a more comprehensive investigation should be made in an attempt to develop a load-carrying structure which would also serve to inclose the building. There is apparently no reason why the load cannot be economically applied directly to the top of a brick wall of suitable materials and workmanship. The steel in

the incased columns undoubtedly served to reinforce the brick masonry, but the question arises whether a number of vertical steel bars of small diameter placed near both faces of the wall would not have provided much greater restraint against lateral buckling of the brick wall and at the same time reduced the cost. The results of further investigations are needed to give a definite answer to this question. Attention should also be given to developing methods for tying the structure supported by the brick wall into the top of the wall so that they will act as a unit under any forces which are likely to come upon them.

The ultimate compressive strength of the bare or unincased steel columns was only 53% of the yield point of the steel, while the maximum compressive load of the incased columns was practically the maximum compressive load for that part of the steel column above the brick masonry, which in turn was, for all practical purposes, the tensile yield point of the steel multiplied by the cross-sectional area of the steel column.

## 1106. DESIGN OF COMBINATION COLUMNS OF STRUCTURAL STEEL AND BRICK MASONRY

It is frequently convenient and economical to use structural steel shapes, particularly the H-column sections, as the steel core of a brick column. The 4″ H-section can be used in the 12¾″ x 12¾″ brick column, while the 8″ H-section fits in exceptionally well with 17″ square brick column. The steel core is completely incased in at least 3″ of masonry. The space between the outer ring of brick and the steel core is filled with clipped brick and grout, or may also be filled with a rich concrete. For combination columns whose slenderness ratio (h/d) exceeds ten, ¼″ or ⅜″ hoops should be placed in every mortar joint.

The American Institute of Steel Construction recommends a maximum compressive stress of 18,000 lbs. per sq. inch for rolled steel in short lengths or where lateral deflection is prevented. The tests at the National Bureau of Standards on steel columns encased in brick masonry indicate that the steel was rigidly supported with negligible deflection.

Therefore, it might be safe to assume an allowable compressive unit stress in the steel core of combination columns as 18,000 lbs. per sq. inch. However, until more comprehensive tests have been made, it is recommended that the allowable compressive stress in the structural steel core be computed according to the following formula, which is similar to that recommended by the American Institute of Steel Construction for bare steel columns, except that the value for "l" has been modified.

$$f_r = \frac{18{,}000}{1 + \frac{(l/r)^2}{18{,}000}} \qquad (16)$$

in which, $f_r$ is the allowable unit stress for structural steel core,

$l$ = unsupported length of column in excess of five feet (h-5),

r = least radius of gyration of section of steel core.

The total permissible load (P) on a combination column of structural steel and brick masonry should not exceed that given by the following formula:

$$P = 0.2\, f_b' A_b + f_r A_r \qquad (17)$$

wherein, $f_b'$ = ultimate compressive strength of brick masonry as determined from test specimens

$A_b$ = net area of the brick masonry section ($A_b = A_g - A_r$)

$A_g$ = gross area of column

$f_r$ = the allowable unit stress in structural steel core as determined by formula (16)

$A_r$ = cross-sectional area of structural steel core.

The maximum height of combination columns is restricted by the maximum value for $l/r$, which should not exceed 120. In other words, the 12¾″ x 12¾″ column with a 4″ H-section steel core is limited to 14′-6″ in height, determined as follows:

The least radius of gyration of a 4″ H-column is 0.95″.

$$l/r = 120$$
$$l = 120 \times 0.95'' = 114''$$
$$\text{maximum height, } h = l + 5'$$
$$= 114'' + 60''$$
$$= 174'' \text{ or } 14'6''$$

In a like manner it will be found that the maximum height for a 17″ square column with 8″ H-section steel core is approximately 27 feet.

**Example:**

What is the permissible load carried by 12¾″ by 12¾″ brick masonry column, 14′ high, with a 4″ H-section structural steel core. The ultimate strength ($f'_b$) of the brick masonry is assumed as 1,800 lbs. per sq. inch.

**Solution:**

$$f_r = \frac{18{,}000}{1 + \frac{(l/r)^2}{18{,}000}}$$
$$l = (14 - 5) \times 12 = 108''$$
$$r = 0.95''$$

Substituting in the above formula $f_r$ is found to be approximately 10,500 lbs. per sq. inch. From formula (17), the total permissible load (P) is:

$$P = 0.2\, f'_b A_b + f_r A_r$$
$$= 0.2 \times 1{,}800\ (162.56 - 3.99) + 10{,}500 \times 3.99$$
$$= 99{,}000 \text{ lbs.}$$

Inasmuch as the slenderness ratio (h/d) of this column exceeds 10, ¼″ hoops, 9¾″ in diameter, will be placed in every mortar joint.

## 1107. COLUMNS WITH LONGITUDINAL REINFORCEMENT

Longitudinal steel rods in brick masonry columns increase the carrying capacity of the column by an amount which compares favorably with the yield point of the steel. Ultimate failure is sometimes due to the buckling of the longitudinal rods, causing the outside shell of the brick masonry to spall or separate from the core. It is recommended, therefore, that at least 1½ in. of mortar or brickwork should cover the rods on the outside and that the vertical rods should be held securely in place by means of bands or tie-rods at intervals not to exceed the least lateral dimension of the column. Such bands or tie-rods cannot be regarded as hooping in the usual sense, because hooping to be sufficiently effective to be considered in the computations must be spaced relatively close.

Practically all longitudinal rods used in R-B-M columns are spliced by overlapping or butt welding. The overlap should be for a sufficient distance to develop the working stress in the reinforcing steel and the lapped rods should be securely wired to each other. Welded splices may be made on butted ends and, when properly made, should develop stress in tension equal to the yield point in the steel. If splices are necessary, it is more convenient to place them immediately above the floor level.

## 1108. COLUMNS WITH HOOPED REINFORCEMENT

The general effect of closely spaced hooping is to promote toughness of the column, increase its ultimate strength and eliminate sudden failure. Thus, it makes the brick masonry column a safer and more reliable structural member, and should permit the use of a somewhat higher working stress.

The theory of hoop reinforcement is roughly summarized as follows:

Brick masonry is like other materials in that compression along one axis is accompanied by a small amount of expansion of the material along perpendicular axes. This is in accord with Poisson's ratio or theory of lateral strains. A column under compression tends to expand laterally. Closely spaced hoops help to hold the column laterally, and thereby develop lateral compressive stresses which in turn tend to neutralize the effect of the longitudinal compressive stresses and thus increase the resistance of the column against failure.

Hoops are preferably circular in shape, although rectangular shapes are frequently used. Hoops may be lap-welded if the strength of the weld is at least equal to the yield point of the rod. A fillet weld has been found preferable to a spot weld. Hoops that are not welded should have overlapping ends bent at least 90° toward the center of the column. These bends should be not less than one inch in diameter with a return of at least three inches. The bends should overlap at least one inch and where practicable, be placed around one of the vertical reinforcing rods. Hoops are seldom spaced more than two courses apart and usually in every course.

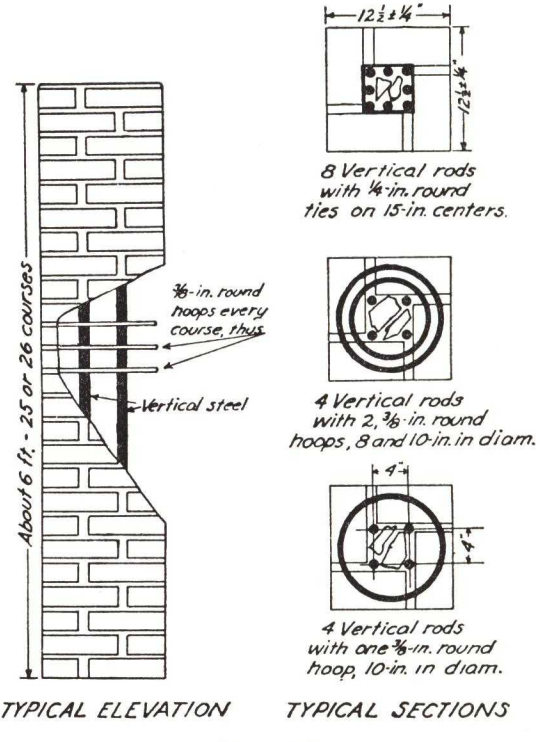



*Figure 30*

*Typical R-B-M columns tested at the University of Wisconsin.*

## 1109. COLUMNS WITH BOTH HOOPED AND LONGITUDINAL REINFORCEMENT

Columns reinforced with hoops and longitudinal rods have the benefit of the effect of both types of reinforcement. The ultimate load is increased beyond the maximum developed with either type separately. Tests indicate that this increase in ultimate strength is approximately equivalent to the sum of the increase in strength of the respective types of reinforcement over the strength of a plain brick masonry column.

## 1110. TESTS ON R-B-M COLUMNS

In the summer of 1933, a series of tests were made on reinforced brick masonry columns at the University of Wisconsin under the direction of Professor M. O. Withey (See A.S.T.M. Proc. Vol. 34, Part II). Thirty-two columns were tested: sixteen with one make of brick and a mortar mix of one part of hydrated lime, three parts of portland cement and twelve parts of sand (1L:3C:12S) by weight; the other sixteen with another make of brick, four of which had the same mortar mix as the first sixteen, while the remaining twelve had a mortar mix of 1L:3C:7½S by weight. Brick prisms were made as typical specimens of each column. These prisms were 8 x 8 x 16 in., made with six courses of brick laid flat as alternate headers and stretchers with half-inch joints, and were tested at the same age as the columns, which was approximately four weeks. The following table summarizes the results of the tests on the mortar, brick and prisms.

**AVERAGE COMPRESSIVE STRENGTH AND MODULI OF ELASTICITY FOR AUXILIARY 8 BY 8 BY 16″ PRISMS**

**Age at Test — 4 weeks**

| Column | Mortar Mix by Weight lime:cement:sand | Modulus of Elasticity Lb./Sq. In. | Compressive Strength Lb./Sq. In. | | |
|---|---|---|---|---|---|
| | | | Mortar | Brick | Prisms |
| Nos. 1 to 16 | 1:3:12 | 905,000 | 2310 | 2734 | 1040 |
| Nos. 17 to 20 | 1:3:12 | 2,680,000 | 2375 | 14420 | 3675 |
| Nos. 21 to 32 | 1:3:7½ | 3,680,000 | 3567 | 14420 | 4521 |

Professor Withey stated that it appeared likely that still stronger prisms and columns can be made with a strong brick, such as used in columns No. 17 to 32, by using a mortar of higher compressive strength.

The columns had varying percentages of longitudinal reinforcement ranging from none to approximately 4% and varying percentages of lateral reinforcement from none to 1½%. Intermediate grade 1 in. round deformed bars were used for longitudinal reinforcement, and ⅜ in. rods for hoops. The columns were approximately 12½″ square. The vertical reinforcement consisted of four or eight 1″ rods, tied with ¼″ rods on 15″ centers, unless lateral reinforcement was used, in which case, ⅜″ hoops, 8 and 10″ in diameter, were placed in every mortar joint. In some columns, a single hoop of the larger diameter was placed in each mortar bed, while in other columns, both sizes of hoops were placed in every mortar bed.

Strain data indicated that the yield point of the longitudinal steel was reached in the test of every R-B-M column. Assuming the steel stress at the ultimate load on the column to be equal to the yield point, the brickwork carried the remainder of the load and the unit stress is calculated accordingly.

Table No. 41 gives a comparison of the strength of columns and prisms as well as the calculated unit stresses in the brickwork and steel of the columns at maximum load.

## TABLE NO. 41

## UNIT STRESSES CARRIED BY BRICKWORK AND BY STEEL IN R-B-M COLUMNS AT MAXIMUM LOAD

| Column | Gross Area, Sq. In. | Core Area, Sq. In. | Percentage of Reinforcement | | Maximum Load | Load at Failure of Core | Unit Stresses | | | Ultimate Strgth. of Prisms Lb. per Sq. In. |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Brickwork | | Steel | |
| | | | Longitudinal | Lateral | | | Based on Gross Area Lb. per Sq. In. | Based on Core Area Lb. per Sq. In. | | |
| | | | % | % | Lb. | Lb. | | | Lb. per Sq. In. | |
| No. 1 | 157.5 | ..... | 0 | 0 | 154800 | ..... | 983 | ..... | ..... | |
| No. 2 | 160.6 | ..... | 0 | 0 | 174900 | ..... | 1090 | ..... | ..... | |
| No. 3 | 162.0 | ..... | 1.92 | 0 | 270000 | ..... | 1667 | ..... | | |
| No. 4 | 161.3 | ..... | 1.93 | 0 | 255000 | ..... | 1580 | ..... | 48720 | |
| No. 5 | 163.2 | ..... | 3.81 | 0 | 412300 | ..... | 2525 | ..... | | |
| No. 6 | 162.6 | ..... | 3.83 | 0 | 428800 | ..... | 2640 | ..... | | |
| No. 7 | 162.6 | 90.8 | 0 | 1.36 | 218000 | 190000 | 1340 | 2090 | ..... | |
| No. 8 | 156.8 | 90.8 | 0 | 1.37 | 175400 | 143000 | 1120 | 1575 | ..... | 1040 |
| No. 9 | 165.8 | 90.8 | 1.88 | 1.32 | 341000 | 323000 | 2060 | 1950 | | |
| No. 10 | 163.2 | 90.8 | 1.90 | 1.34 | 333500 | 320500 | 2040 | 1920 | | |
| No. 11 | 161.3 | 90.8 | 3.86 | 1.36 | 467000 | 378000 | 2900 | 874 | | |
| No. 12 | 163.8 | 90.8 | 3.80 | 1.33 | 453000 | ..... | 2765 | ..... | 48720 | |
| No. 13 | 160.0 | 90.8 | 1.94 | 0.75 | 343500 | ..... | 2145 | ..... | | |
| No. 14 | 161.3 | 90.8 | 1.93 | 0.75 | 322200 | 269000 | 2000 | 1335 | | |
| No. 15 | 162.5 | 90.8 | 3.83 | 0.74 | 461750 | 420000 | 2840 | 1370 | | |
| No. 16 | 162.0 | 90.8 | 3.84 | 0.75 | 460400 | 389250 | 2840 | 1010 | | |
| No. 17 | 144.7 | ..... | 0 | 0 | 361500 | ..... | 2500 | ..... | ..... | |
| No. 18 | 144.5 | ..... | 0 | 0 | 377500 | ..... | 2615 | ..... | ..... | 3675 |
| No. 19 | 150.0 | ..... | 4.15 | 0 | 612700 | ..... | 4080 | ..... | | |
| No. 20 | 151.9 | ..... | 4.10 | 0 | 631500 | ..... | 4160 | ..... | 48720 | |
| No. 21 | 148.8 | 90.8 | 0 | 1.54 | 581000 | 456000 | 3900 | 5030 | ..... | |
| No. 22 | 150.7 | 90.8 | 0 | 1.51 | 564600 | 449000 | 3740 | 4950 | ..... | |
| No. 23 | 150.7 | 90.8 | 2.07 | 1.51 | 608000 | 560500 | 4040 | 4655 | | |
| No. 24 | 150.7 | 90.8 | 2.07 | 1.52 | 605500 | 538500 | 4020 | 4410 | | |
| No. 25 | 152.0 | 90.8 | 4.10 | 1.50 | 768700 | 694000 | 5050 | 4605 | | |
| No. 26 | 154.5 | 90.8 | 4.03 | 1.47 | 745000 | 585000 | 4820 | 3320 | 48720 | 4521 |
| No. 27 | 156.8 | 90.8 | 1.98 | 1.48 | 620000 | 515500 | 3950 | 4150 | | |
| No. 28 | 159.4 | 90.8 | 1.95 | 1.46 | 645000 | 484000 | 4045 | 3785 | | |
| No. 29 | 152.5 | 90.8 | 2.04 | 1.52 | 591500 | 510500 | 3880 | 4090 | | |
| No. 30 | 162.0 | ..... | 0 | 0 | 542000 | ..... | 3340 | ..... | ..... | |
| No. 31 | 158.4 | 90.8 | 1.96 | 1.41 | 620000 | 565500 | 3915 | 4715 | | |
| No. 32 | 154.6 | 90.8 | 2.01 | 0.80 | 712200 | 440500 | 4610 | 3290 | 48720 | |

## TABLE NO. 42

The unit stresses in the steel and brickwork were computed at one-fourth the ultimate load. The values of the secant modulus of elasticity were calculated at one-fourth the ultimate load and also after repetition of a load of 34 to 5% of the maximum. These values are listed in the following table:

| Column | Percentage of Reinforcement | | Ultimate Strgth. | Unit Stress at ¼ Ultimate Load | | | Prism Strgth. | Modulus of Elasticity of Brickwork at ¼ Ultimate Load Lb. per Sq. In. | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Longitudinal | Lateral | | In Gross Area | In Brickwork | In Steel | | Column | Column after Set | Prism |
| | % | % | Lb./ Sq. In. | Lb./ Sq. In. | Lb./ Sq In. | Lb./ Sq. In. | Lb. per Sq. In. | | | |
| No. 1 | 0 | 0 | 983 | 246 | 246 | ..... | | 1410000 | ..... | |
| No. 2 | 0 | 0 | 1090 | 273 | 273 | ..... | | 1610000 | 1760000 | |
| No. 3 | 1.92 | 0 | 1667 | 417 | 294 | 6570 | | 1910000 | ..... | |
| No. 4 | 1.93 | 0 | 1580 | 395 | 290 | 4730 | | 2070000 | ..... | |
| No. 5 | 3.81 | 0 | 2525 | 631 | 365 | 7350 | | 2580000 | ..... | |
| No. 6 | 3.83 | 0 | 2640 | 660 | 354 | 8340 | | 2380000 | 2160000 | |
| No. 7 | 0 | 1.36 | 1340 | 335 | 335 | ..... | | 1430000 | 1370000 | |
| No. 8 | 0 | 1.37 | 1120 | 280 | 280 | ..... | 1040 | 1110000 | 940000 | 905000 |
| No. 9 | 1.88 | 1.32 | 2060 | 515 | 370 | 8070 | | 1920000 | 1860000 | |
| No. 10 | 1.90 | 1.34 | 2040 | 510 | 356 | 8490 | | 1800000 | 1650000 | |
| No. 11 | 3.86 | 1.36 | 2900 | 725 | 369 | 9570 | | 2270000 | 2240000 | |
| No. 12 | 3.80 | 1.33 | 2765 | 691 | 336 | 9690 | | 2140000 | 2110000 | |
| No. 13 | 1.94 | 0.75 | 2145 | 536 | 388 | 8070 | | 1990000 | 1990000 | |
| No. 14 | 1.93 | 0.75 | 2000 | 500 | 370 | 7080 | | 2120000 | 1980000 | |
| No. 15 | 3.83 | 0.74 | 2840 | 710 | 376 | 9060 | | 2350000 | 2220000 | |
| No. 16 | 3.84 | 0.75 | 2840 | 710 | 294 | 11130 | | 1910000 | 2110000 | |
| No. 17 | 0 | 0 | 2500 | 625 | 625 | ..... | | 3000000 | ..... | |
| No. 18 | 0 | 0 | 2615 | 654 | 654 | ..... | | 3500000 | 3550000 | 2680000 |
| No. 19 | 4.15 | 0 | 4080 | 1020 | 750 | 7260 | 3675 | 4210000 | 3800000 | |
| No. 20 | 4.10 | 0 | 4160 | 1040 | 779 | 7140 | | 4370000 | 3800000 | |
| No. 21 | 0 | 1.54 | 3800 | 975 | 975 | ..... | | 3810000 | 3920000 | |
| No. 22 | 0 | 1.51 | 3740 | 935 | 935 | ..... | | 3520000 | 3680000 | |
| No. 23 | 2.07 | 1.51 | 4040 | 1010 | 878 | 7200 | | 4200000 | 4150000 | |
| No. 24 | 2.07 | 1.52 | 4020 | 1005 | 804 | 10530 | | 2860000 | 3050000 | |
| No. 25 | 4.10 | 1.50 | 5050 | 1262 | 908 | 9600 | | 3940000 | 4060000 | |
| No. 26 | 4.03 | 1.47 | 4820 | 1205 | 799 | 10830 | 4521 | 3340000 | 3630000 | 3680000 |
| No. 27 | 1.98 | 1.48 | 3950 | 987 | 863 | 7200 | | 4100000 | 3730000 | |
| No. 28 | 1.95 | 1.46 | 4045 | 1011 | 867 | 8200 | | 3690000 | 3350000 | |
| No. 29 | 2.04 | 1.52 | 3880 | 970 | 819 | 8220 | | 3540000 | ..... | |
| No. 30 | 0 | 0 | 3340 | 835 | 835 | ..... | | 3270000 | ..... | |
| No. 31 | 1.96 | 1.41 | 3915 | 979 | 821 | 8850 | | 3310000 | 3600000 | |
| No. 32 | 2.01 | 0.80 | 4610 | 1153 | 998 | 8640 | | 4000000 | 4060000 | |

The compressive yield point of the longitudinal steel was 48,720 lbs./sq. in.
The tensile yield point of the lateral steel was 53,800 lbs./sq. in.

Quoting from Professor Withey's paper:

"The tests show that the effective strength of a reinforced brick masonry column equals the sum of three components, the strength of the plain masonry, the strength of the longitudinal steel at its yield point, and a less certain one due to lateral restraint offered by hoops when placed in the horizontal joints. A formula involving these components which is in reasonable agreement with the results is presented.

$$\frac{P}{A} = f_b (1\text{-}p) + pf_s + Kp'f'_s$$

Where, $P$ = maximum load,
$A$ = gross area of cross-section,
$f_b$ = unit load for plain brick column,
$p$ = longitudinal steel ratio in terms of gross area,
$p'$ = lateral steel ratio in terms of gross area,
$f_s$ = yield point of longitudinal steel,
$f'_s$ = yield point of hoops, or weld strength if less than yield point and,
$K$ = constant tentatively assumed to depend upon the ratio $A/A_c$ and possibly on the kind of brick, where $A_c$ = area of core within outer hoop.

"For these tests with $A/A_c$ approximately 1.7, $f'_s$ is 54,000 lbs. per sq. in., $p'$ is 0.0075 for columns No. 1 to 16 and 0.015 for columns No. 17 to 32, and K is approximately 0.6. The validity of the last term of the equation is questionable until more data on the action of hoops have been secured.

"The data indicate that with very strong brick and mortar, a ratio of gross area to core area of 1.7, 2½% of lateral reinforcement and 7% of longitudinal reinforcement based on core area (approximately 4% based on gross area), that crushing strengths of 5000 lbs. per sq. in. may be obtained on gross area and 7000 lbs. per sq. in. on core area alone.

"The addition of ¾% of hoops based on gross area promoted toughness, increased the strength and eliminated sudden failures in these tests. It seems probable that for ratios of $A/A_c$ less than 1.5, the percentage of hooping or other desirable lateral reinforcement could be reduced to 0.6%.

"With careful workmanship including filled joints and good materials, reinforced brick masonry columns should be safe under static loads with a factor of safety of 4. For columns of high-strength brick and mortar, reinforced with both longitudinal steel and hoops, a factor of safety of 3½ is probably safe."

"Results of tests on 33 R-B-M columns made at Lehigh University in 1933 under the direction of Professor Inge Lyse indicate a remarkable similarity. (See Journal, American Ceramic Society, Nov. 1933; also Eng. News-Record, March 16, 1933 and Jan. 4, 1934). Professor Lyse recommended a similar design formula with a safety factor of 4.0. In commenting on the tests made at both universities, Professor Lyse states: "It is gratifying to find such complete agreement between investigations carried out at two different places, which adds weight to the conclusion that reinforced brick columns of proper design and workmanship may well be used in modern engineering construction."

## 1111. FORMULAS FOR SAFE WORKING LOADS

### (a) Columns with Slenderness Ratio not greater than 10

A formula similar to that proposed by Professor Withey is used to compute the allowable load on an R-B-M column. The nomenclature of symbols used by Professor Withey are not quite the same in all instances as the standard notations used in this volume. In the following equation, which is recommended for determining the safe

working loads on R-B-M columns having slenderness ratios of not more than ten, the basic principles of the Withey formula remain, but safety factors have been added and the nomenclature has been changed to correspond with that generally used in this book.

$$\frac{P}{A_g} = 0.20f'_b\,(1\text{-}p_g) + 0.6f_s p_g + 0.1p'f'_s\,A_c/A_g \quad \ldots\ldots\ldots\ldots\ldots (18)$$

in which P = permissible axial load on a column

$A_g$ = gross area of column

$f'_b$ = compressive strength of brick masonry as determined by test specimens

$p_g$ = ratio of the cross-sectional area of longitudinal reinforcement to the gross area ($A_g$) of column

$f_s$ = nominal working stress in longitudinal (vertical) reinforcement, usually taken as 40% of the specified yield point

$p'$ = ratio of volume of lateral reinforcement to volume of brick masonry core

$f'_s$ = useful limit stress of lateral reinforcement

$A_c$ = area of core of column, measured to outside of lateral reinforcement.

A safety factor of 3½ or 4 has been recommended for R-B-M columns. In comparing the above equation for safe working loads with the Withey formula for ultimate strengths, it is found that the first term of Equation (18), representing the safe unit load on the plain masonry, has a safety factor of 5; the second term, representing the unit load taken by the longitudinal reinforcement, has a safety factor slightly in excess of 4; while the last term, representing the effect of lateral reinforcement, has a safety factor of 10 when compared with the constant used in the Withey formula for the column tests at University of Wisconsin.

It is advisable to maintain this fairly high safety factor in the third term until sufficient additional column tests are made to ascertain more definitely the full effect and action of lateral reinforcement. The equation may be written also in the following form:

$$P = 0.2\,A_g f'_b \times (1\text{-}p_g) + 0.6f_s p_g\,A_g + 0.1p'f'_s A_c \quad \ldots\ldots\ldots\ldots\ldots (19)$$

The three terms of the R-B-M column formula represent the three components which comprise the effective strength of a reinforced brick masonry column; namely, the strength of the brick masonry, the strength of the vertical or longitudinal steel, and the additional value due to the support or lateral restraint of the hoops.

### (b) Columns With Slenderness Ratio Between 10 and 20

Equations (18) and (19) apply to columns whose unsupported length is not greater than ten times the least lateral dimension. There has been very little investigation of long columns (ratio of length to least width in excess of ten), but the similarity of reinforced brick masonry to reinforced concrete is such that a similar formula is proposed which reduces the permissible axial load on columns as the slenderness ratio (h/d) increases. The permissible axial loads (P′) for long R-B-M columns (slender ratios between 10 and 20) is determined by Formula (20).

$$P' = P(1.3\text{–}.03h/d) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

in which P′ = total allowable, or permissible, axial load on a long column

P = total allowable, or permissible, axial load on a short column

h = unsupported length of the column

d = the least lateral dimension of the column.

## TABLE NO. 43

### FOR USE WITH R-B-M COLUMNS 12¾″ x 12¾″

**Gross area ($A_g$), 162.56 sq. in.**

**Maximum Unsupported Height — 10′-7½″**

Values for $[0.20f'_b\ (1-p_g)\ A_g]$ and $[0.6f_sp_gA_g]$

| Longitudinal Reinforcement | | | Net area of Brickwork | Values of $0.20f'_b\ (1-p_g)\ A_g$ in Lbs. when $f'_b$ equals | | | | | | Values of $0.6f_sp_gA_g$ in lbs. when $f_s$ equals | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Ratio $p_g$ % | No. and size of bars | Area of steel $A_s$ Sq. In. | $(1-p_g)A_g$ | 1200 | 1500 | 1800 | 2100 | 2400 | 3000 | 16000 | 18000 | 20000 |
| 0.96 | 8–½″Ø | 1.57 | 160.99 | 38640 | 48300 | 57960 | 67620 | 77280 | 96590 | 15080 | 16970 | 18840 |
| 1.09 | 4–¾″Ø | 1.77 | 160.79 | 38590 | 48240 | 57880 | 67530 | 77180 | 96470 | 16990 | 19120 | 21240 |
| 1.36 | 4–¾″Ø<br>4–⅜″Ø | 2.21 | 160.35 | 38480 | 48100 | 57720 | 67350 | 76970 | 96210 | 22200 | 23870 | 26520 |
| 1.57 | 4–¾″Ø<br>4–½″Ø | 2.56 | 160.00 | 38400 | 48000 | 57600 | 67200 | 76800 | 96000 | 24580 | 27650 | 30720 |
| 1.93 | 4–1″Ø | 3.14 | 159.42 | 38260 | 47830 | 57390 | 66960 | 76520 | 95650 | 30150 | 33910 | 37680 |
| 2.17 | 8–¾″Ø | 3.53 | 159.03 | 38170 | 47710 | 57250 | 66790 | 76330 | 95420 | 33900 | 38120 | 42360 |
| 2.42 | 4–1″ Ø<br>4–½″Ø | 3.93 | 158.63 | 38070 | 47590 | 57110 | 66620 | 76140 | 95180 | 37700 | 42440 | 47160 |

Values for $0.1\ p'f'_sA_c$

| Lateral Hoops | | | | $A_c$ | Values of $0.1\ p'f'_s{}'A_c$ in lbs. when $f'_s$ equals | | | |
|---|---|---|---|---|---|---|---|---|
| Diameter of rods | Diameter of Hoops | $p'$ % | Spacing | Sq. In. | 35000 | 40000 | 45000 | 50000 |
| ¼″ | 9¾″ | 0.70 | 2¾″ | 78.5 | 1925 | 2200 | 2475 | 2750 |
| ¼″ | 6¾″ & 9¾″ | 1.18 | 2¾″ | 78.5 | 3245 | 3710 | 4175 | 4640 |
| ⅜″ | 9¾″ | 1.48 | 2⅞″ | 80.3 | 4170 | 4760 | 5350 | 5940 |
| ⅜″ | 6¾″ & 9¾″ | 2.49 | 2⅞″ | 80.3 | 7000 | 8000 | 9000 | 10000 |

## TABLE NO. 44 — FOR USE WITH R-B-M COLUMNS 17″ x 17″

**Gross area ($A_g$) = 289 sq. in. — Maximum Unsupported Height — 14′-2″**

Values for $[0.20f'_b\ (1\text{-}p_g)\ A_g]$ and $[0.6f_sp_gA_g]$

| Longitudinal Reinforcement | | | Net area of Brickwork | Values of $0.20f'_b\ (1\text{-}p_g)\ A_g$ in Lbs. when $f'_b$ equals | | | | | | Values of $0.6f_sp_gA_g$ in lbs. when $f_s$ equals | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Ratio $p_g$ % | No. and size of bars | Area of steel $A_s$ Sq. In. | $(1\text{-}p_g)A_g$ | 1200 | 1500 | 1800 | 2100 | 2400 | 3000 | 16000 | 18000 | 20000 |
| 0.54 | 8–½″Ø | 1.57 | 287.43 | 68980 | 86230 | 103475 | 120720 | 137970 | 172460 | 15080 | 16970 | 18840 |
| 0.61 | 4–¾″Ø | 1.77 | 287.23 | 68930 | 86170 | 103400 | 120640 | 137870 | 172340 | 16990 | 19120 | 21240 |
| 0.69 | 8–½″□ | 2.00 | 287.00 | 68800 | 86100 | 103320 | 120540 | 137760 | 172200 | 19200 | 21600 | 24000 |
| 0.85 | 8–⅝″Ø | 2.45 | 286.55 | 68770 | 85965 | 103160 | 120350 | 137540 | 171930 | 23500 | 26450 | 29400 |
| 1.08 | 4–¾″Ø<br>4–⅝″Ø | 3.00 | 286.00 | 68640 | 85800 | 102960 | 120120 | 137280 | 171600 | 28800 | 32400 | 36000 |
| 1.08 | 12–½″□ | 3.00 | 286.00 | 68640 | 85800 | 102960 | 120120 | 137280 | 171600 | 28800 | 32400 | 36000 |
| 1.22 | 8–¾″Ø | 3.53 | 285.47 | 68510 | 85640 | 102770 | 119900 | 137030 | 171280 | 33900 | 38150 | 42400 |
| 1.27 | 12–⅝″Ø | 3.68 | 285.32 | 68480 | 85600 | 102720 | 119840 | 136950 | 171190 | 35400 | 39800 | 44200 |
| 1.38 | 16–½″□ | 4.00 | 285.00 | 68400 | 85500 | 102600 | 119700 | 136800 | 171000 | 38400 | 43200 | 48000 |
| 1.70 | 16–⅝″Ø | 4.91 | 284.09 | 68180 | 85230 | 102270 | 119320 | 136360 | 170450 | 47100 | 53000 | 58900 |
| 1.83 | 12–¾″Ø | 5.30 | 283.70 | 68090 | 85110 | 102130 | 119160 | 136170 | 170220 | 50900 | 57300 | 63700 |
| 2.17 | 8–1″Ø | 6.28 | 282.72 | 67860 | 84820 | 101780 | 119750 | 135700 | 169630 | 60300 | 67850 | 75400 |
| 2.44 | 16–¾″Ø | 7.06 | 281.94 | 67670 | 84580 | 101500 | 119420 | 135330 | 169160 | 67800 | 76300 | 84800 |
| 2.77 | 8–1″□ | 8.00 | 281.00 | 67440 | 84300 | 101160 | 118020 | 134880 | 168600 | 76800 | 86400 | 96000 |

Values for $0.1\ p'f'_sA_c$

| Lateral Hoops | | | $A_c$ | Values of $0.1\ p'f'_sA_c$ in lbs. when f′ equals | | | |
|---|---|---|---|---|---|---|---|
| Diameter of rods | Diameter of hoops | $p'$ % | Sq. In. | 35000 | 40000 | 45000 | 50000 |
| ¼″ | 14″ | 0.49 | 159.6 | 2740 | 3130 | 3520 | 3910 |
| ¼″ | 11″ & 14″ | 0.88 | 159.6 | 4920 | 5620 | 6320 | 7020 |
| ⅜″ | 14″ | 1.05 | 162.0 | 5950 | 6800 | 7650 | 8500 |
| ⅜″ | 11″ & 14″ | 1.87 | 162.0 | 10600 | 12100 | 13600 | 15100 |

## 1112. TABLES OF DESIGN DATA FOR TWO TYPICAL R-B-M COLUMNS

In order to simplify the design of two typical R-B-M columns, 12¾" and 17" square, tables (pg. 252, 253) have been computed, giving the values of each of these three terms for various strengths of brick masonry and steel reinforcement as well as varying percentages of reinforcing steel. Tables No. 43 and 44 contain data for use in the design of short columns (h/d not more than 10) and Table No. 45 contains the correction factor to be applied in the design of columns whose slenderness ratio (h/d) is greater than 10 but not more than twenty. Examples are given in Section 1113 to illustrate their use.

The following table gives values for the correction factor (1.3-.03 h/d) and corresponding reciprocals to be used in connection with the design of long columns. Examples in the use of these tables are given in Section 1113.

### TABLE NO. 45

### CORRECTION FACTOR (1.3–.03 h/d) AND RECIPROCAL TO BE USED IN THE DESIGN OF LONG COLUMNS

**12¾" x 12¾" Columns**

| Height of Column | 10'-7½" | 12'-0" | 14'-0" | 16'-0" | 18'-0" | 20'-0" |
|---|---|---|---|---|---|---|
| Correction Factor | 1.000 | 0.961 | 0.905 | 0.848 | 0.792 | 0.735 |
| Reciprocal | 1.000 | 1.040 | 1.105 | 1.180 | 1.262 | 1.361 |

**17" x 17" Columns**

| Height of Column | 14'-2" | 16'-0" | 18'-0" | 20'-0" | 22'-0" | 24'-0" | 26'-0" | 28'-0" |
|---|---|---|---|---|---|---|---|---|
| Correction Factor | 1.000 | 0.961 | 0.919 | 0.877 | 0.834 | 0.792 | 0.749 | 0.707 |
| Reciprocal | 1.000 | 1.040 | 1.088 | 1.14 | 1.200 | 1.262 | 1.335 | 1.414 |

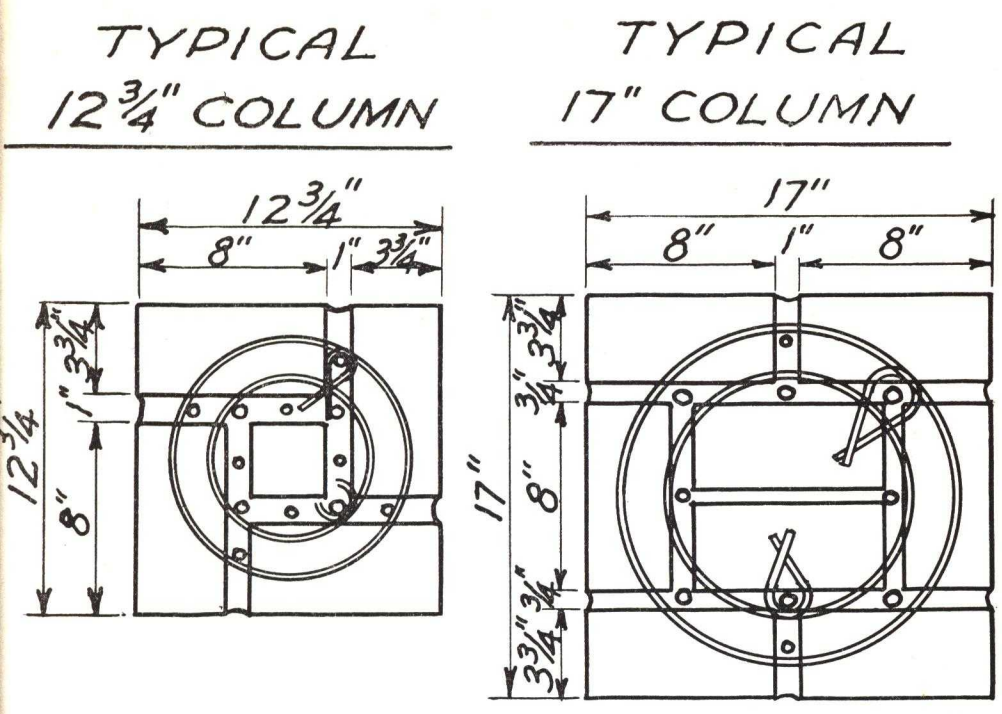
## 1113. THE USE OF TABLES IN COLUMN DESIGN

**Example 1 (Short Column):**

Design an R-B-M column, 10 feet high, to support safely an axial load of 100,000 lbs. The ultimate compressive strength of the brick masonry ($f_b'$) is assumed at 2100 lbs. per sq. inch; $f_s$ for the vertical steel is 16,000 lbs. per sq. inch and $f_s'$ for the hoop steel is 40,000 lbs. per sq. inch.

**Solution:**

The short column should have its least width not less than one-tenth the height, or 12 inches. Therefore, a 12¾" square column will be investigated first and reference made to the tables for this size column. The values for the third term ($0.1\ p'f_s'A_c$) are relatively small, so it will be necessary for the sum of the first two terms to equal practically the total given load. A glance down the column under $f_b' = 2100$ in Table No. 43 for values of the first term and under $f_s = 16{,}000$ for values of the second term will




reveal that several percentages or arrangements of vertical steel will give values whose sums approximate 100,000. If $p_g$ is 1.93%, the sum of 66,960 and 30,150 equals 97,110 lbs., while a $p_g$ of 2.17% gives a total of 100,690 pounds. Now, refer to the table of values for the third term ($0.1\ p'f_s'A_c$), under column $f_s' = 40{,}000$ and it is noted that a $\frac{3}{8}''$ hoop with p′ of 1.48% has a value of 4,760 lbs. Adding 4,760 lbs. to 97,100 lbs. and it is found to be 101,860 lbs. This is more than the 100,000 lbs. requirement of the problem. It is noted, however, that a smaller percentage of vertical steel ($p_g = 2.56\%$) will reduce the sum of the first two terms to 91,780 lbs., which with the additional value of the third term falls below the required 100,000 lbs. Instead of the $\frac{3}{8}''$ hoops, we might have used two $\frac{1}{4}''$ hoops, having diameters of $6\frac{3}{4}''$ and $9\frac{3}{4}''$ and p′ of 1.18%. The value for the term $0.1p'f_s'A_c$ becomes 3,710 lbs., which added to our 97,100 lbs. equals 100,810 lbs. This would also satisfy the requirements of the problem and the mortar beds need be only $\frac{1}{4}'' + \frac{1}{8}'' = \frac{3}{8}''$ thick. It is usually more convenient to place only one hoop in a mortar bed, and therefore, in this case the single $\frac{3}{8}''$ hoop is recommended, and we shall use a $12\frac{3}{4}''$ x $12\frac{3}{4}''$ column with four 1″ round bars vertically and $\frac{3}{8}''$ hoops, having a diameter of $9\frac{3}{4}''$, placed in every mortar joint. The vertical joints will be as shown on the standard plan, while the horizontal mortar beds may be either $\frac{1}{2}''$ or $\frac{5}{8}''$ thick.

In this problem, only four vertical rods are required and will be placed at the four corners of the center bat or half brick. In columns where eight vertical rods may be required, the larger rods are placed at these same four corners with the other four rods in between. This permits the outside layer of brick to rotate a quarter turn with each course, and thereby creating a better bonding appearance.

With eight rods around the inside square, the four outside brick are laid up and kept a few courses in advance of that part inside the reinforcement bars. This inner square is then filled with grout and brick bats (small pieces of brick), but always in short lifts after the mortar in the joints of the outside brick has partially hardened.

**Example 2 (Long Column):**

Design on R-B-M column with an unsupported height of 24 feet to carry an axial load of 108,000 lbs. Compressive tests on brick prisms 8″ square and 24″ high indicate an ultimate compressive strength ($f'_b$) for the brick masonry in excess of 1800 lbs. per sq. inch; $f_s$ for the vertical steel is assumed as 16,000 lbs. per sq. inch and $f'_s$ for the hoop steel is 40,000 lbs. per sq. inch.

**Solution:**

Since it is recommended that the slenderness ratio of an R-B-M column be not more than 20, the 12¾″ square column is eliminated from consideration. From Table 45, the correction factor, based on (1.3-.03h/d), for a 17″ x 17″ R-B-M column 24 ft. high, is 0.792 and its reciprocal 1.262. If we divide the given load (P′ = 108,000 lbs.) by the factor 0.792, or multiply by the reciprocal 1.262, the result (136,300 lbs.) is the equivalent loading (P) required for computations using the tables for short columns.

The design is now identical to that of a short column to support a load of 136,300 lbs. From the tables for 17″ x 17″ columns, under columns for $f'_b$ = 1800 and $f_s$ = 16,000, it is found that eight ⅝″ round bars ($p_g$ = 0.85%) give values of 103,160 and 23,500 lbs. or a sum of 126,660 lbs. There remains 136,300 minus 126,660 = 9,640 lbs. to be taken by the lateral reinforcement (hoops). The table for this term indicates that two ⅜″ hoops (p′ = 1.87%) has a value of 12,100. The total supporting value (P) of the short column is 103,160 + 23,500 + 12,100 = 138,760 lbs., which is ample for the required amount of 136,300 lbs.

As an additional check, we may refer to formula (17) P′ = P (1.3-.03h/d) and solve for the permissible axial load (P) on a long column.

$$\begin{aligned} P' &= P(1.3-.03h/d) \\ &= 138{,}760\ (1.3-.03 \times 24 \times 12/17) \\ &= 138{,}760\ (1.3-0.508) \\ &= 109{,}900 \text{ lbs.} \end{aligned}$$

The original problem gave 108,000 lbs. as the load to be supported. Therefore, the column as designed will be acceptable.

## 1114. MISCELLANEOUS RECOMMENDATIONS FOR R-B-M COLUMNS

Principal R-B-M columns should have a minimum thickness of 12 inches and a minimum gross area of 144 sq. in. Posts or columns that are not continuous from story to story may have a minimum thickness of 8 in.

For columns built as part of walls or piers, the gross area ($A_g$) of the column section should be taken as a circle, square or rectangle of which the sides are 1½ in. outside the lateral reinforcement. It is permissible to design a circular column and build it as a square column with a side equal to the diameter of the circular column.

The unsupported length of columns is taken as the clear distance between the floor surface and the under side of the deeper beam framing into the column at the next higher floor level.

The ratio ($p_g$) of vertical or longitudinal reinforcement is preferably not less than 0.50%, nor more than 6% of the gross area ($A_g$). The vertical bars should be not less than four in number, and not less than ⅜ in. diameter. The reinforcement should be held firmly in its designed position and protected by a covering of masonry not less than 1½ in. thick.

Lateral reinforcement is made with rods, not less than ¼″ size, usually shaped in a circle with the ends united or overlapped in such a way as to develop the full strength of the rod. The maximum diameter of these hoops is 3 in. less than the least lateral dimension of the column. Hoops are usually placed in every course, making the centering



Courtesy National Bureau of Standards

*Figure 31*

*Steel column incased in brick masonry being removed from testing machine after buckling of the short unincased section.*

approximately 2¾ inches. When more than one hoop is used in each course, the differential in the diameter of the hoops is approximately 3 inches. The ratio (p') of the volume of lateral reinforcement to volume of brick masonry core should be not less than 0.50% and preferably 1.0 per cent or more.

Until more complete test data are available regarding the effect of eccentric loading on R-B-M columns, it is recommended that the extreme fiber stress in a column due to eccentric loading be computed by formula (15). If the maximum compressive stress exceeds the allowable unit stress in the masonry either the size of the column or the amount of reinforcing steel should be increased, or both. If there is a probability of tension developing in the longitudinal bars of the column, an adequate connection must be made by overlapping or welding the ends of the bars to take this tension.

## 1115. CONSTRUCTION PROCEDURE

In building R-B-M columns, it is important that dowels of proper length be placed accurately in the footings. After the footings are completed, the vertical bars in the column are securely held in place by wire ties and frequently a wood template is used to hold the rods at the top. Before the uppermost ends of the rods are tied or braced in place, it is sometimes preferable that the required number of hoops be slipped over the top of the vertical bars. This is essential if the hoops are welded. These hoops are held temporarily near the top, until removed as needed for placement in the respective mortar beds. It is not necessary to wire the hoops to the longitudinal reinforcement, but care should be observed in placing each hoop so that it is centered on the column section and the core of the column is straight and plumb.

The brickwork is similar in character to other forms of reinforced brick masonry. A full bed of mortar is spread evenly, the lateral reinforcement imbedded therein and the mortar again trowelled smooth before the next course of brick is laid. The brick are placed firmly in the mortar, usually by "shoving", with the exterior of all vertical joints filled solidly with mortar which had been buttered on the brick before laying. Brick bats or large pieces may be used inside or around the vertical reinforcement wherever it is impossible or inconvenient to lay full-size brick. Tests prove that these bats, surrounded by grout, develop strength comparable with good masonry.

After all brick are laid in a course, with all exterior vertical joints sealed with mortar, as previously described, grout is poured so as to fill completely all remaining space. The large pieces or bats of brick are then "floated" or placed in the grout and tapped or pressed firmly down to insure their proper embedment. A little precaution may be necessary to insure the complete filling around reinforcement bars. The next bed of mortar is then spread and the process repeated.

Sometimes, the inner core is filled with a rich concrete. In such cases, the brickwork is completed several hours, and preferably a whole day, before the concrete is poured into the core. Needless to say, the concrete should be well-tamped to insure complete filling of voids and complete encasement of the reinforcement.

# CHAPTER 12

# FOOTINGS, FOUNDATIONS AND RETAINING WALLS

## 1201. FOOTINGS, GENERAL

The area of a footing must be such that the safe bearing capacity of the supporting soil is not exceeded. It is difficult to eliminate all settlement, but the footings and their unit pressure should be planned so that any settlement which may occur will be uniform over the whole structure.

The total load used to design footings consists of the load used in designing the column or wall, plus the weight of the column or wall and the footing. In the computations for moments and shears, the upward reaction is frequently based on the total design load on the column or wall plus the weight of the column or wall, without allowing for the compensating effect of the weight of the footing.

The critical section for bending moment in a footing which supports a column, pedestal or wall is usually considered to be at the face of the column, pedestal or wall. For footings under metallic bases, the critical section is assumed to be midway between the face of the column and the edge of the metallic base. The critical section for diagonal tension is taken at a distance from the face of the wall or column equal to the effective depth of the footing. It is customary, however, and considered good practice to make the depth of small footings sufficient to eliminate any necessity for diagonal tension reinforcement. The critical section for bond stress in the reinforcement rods is taken in the plane at the face of the wall. Allowable unit shear and bond stresses are given in Chapter 9.

## 1202. WALL FOOTINGS

Wall Footings are designed as cantilever slabs projecting on each side of the wall. The reinforcement for bending moment in a wall footing consists of bars placed at right angles to the face of the wall in the horizontal mortar bed near the bottom of the footing. Some architects and engineers place a small amount of reinforcement in the footing parallel to the face of the wall. This is recommended where there is a possibility of unequal settlement of the supporting soil or in certain regions where the clay soils are subject to cracks during dry periods.

### Illustrative Problem

Design a footing to support a 20″ brick wall having a total vertical load of 30,000 lbs. per foot. Assume $f_b = 600$; $f_s = 16{,}000$; $n = 18$.

Bearing value of soil (Firm, dry clay) is 3 tons per sq. foot.

### Solution

Referring to Table 80, for $n = 18$, $f_b = 600$ and $f_s = 16{,}000$, we find $p = .0076$; $j = 0.866$ and $K = 104.66$. Width of footing is $\frac{30{,}000}{2000 \times 3} = 5$ feet.

It will be assumed that the upward thrust of 6000 lbs. per sq. foot is the load causing moment in the footing. The weight of footing actually acts in the opposite direction, tending to reduce the moment, but, as an additional factor of safety, will not be considered.

The maximum moment for a cantilever beam is at the support, which in this instance is the face of the wall.

$$M = \frac{wl^2}{2} = \frac{6000 \times 1.67^2 \times 12}{2} = 100{,}000 \text{ in-lb.}$$

$$bd^2 = \frac{M}{K} = \frac{100{,}000}{104.66} = 955$$

since, $b = 12$

$$d^2 = \frac{956}{12} = 79.6$$

$$d = 8.93''$$

The effective depth (d) will be actually $2\frac{1}{4} + \frac{1}{2} + 3\frac{3}{4} + \frac{1}{2} + 2\frac{1}{4} + \frac{1}{2} = 9\frac{3}{4}''$.

**Reinforcement**

$A_s = pbd = .0076 \times 12 \times 8.93 = 0.815$ sq. in. $\frac{5}{8}''$ round rods at $4\frac{1}{2}''$ spacing equals 0.819 sq. in.

**Shear** at $9\frac{3}{4}''$ from wall:

$$V = \frac{6000 \times (20\text{-}9\frac{3}{4})}{12} = 5125 \text{ lb.}$$

$$\text{Unit shear, } v = \frac{V}{bjd} = \frac{5125}{12 \times .866 \times 9\frac{3}{4}} = 50.5 \text{ lbs./sq. in.}$$

According to Section 916, the allowable value for shear is 35 lbs. per sq. inch for footings where reinforcing bars have no special anchorage. To avoid special reinforcement for diagonal tension, it will be necessary to increase the effective depth of the footing to reduce the shearing stress to within the allowable amount.

Inasmuch as shear is frequently the factor in determining the effective depth of an R-B-M footing, the following formula may be used to check or obtain the effective depth (d) of a footing in which the shearing stress will not exceed the allowable values.

$$d = \frac{wl}{12jv + w/12}$$

wherein, w = upward pressure per sq. ft. on footing,

$l$ = distance in feet that footing projects beyond face of wall,

v = allowable unit shear.

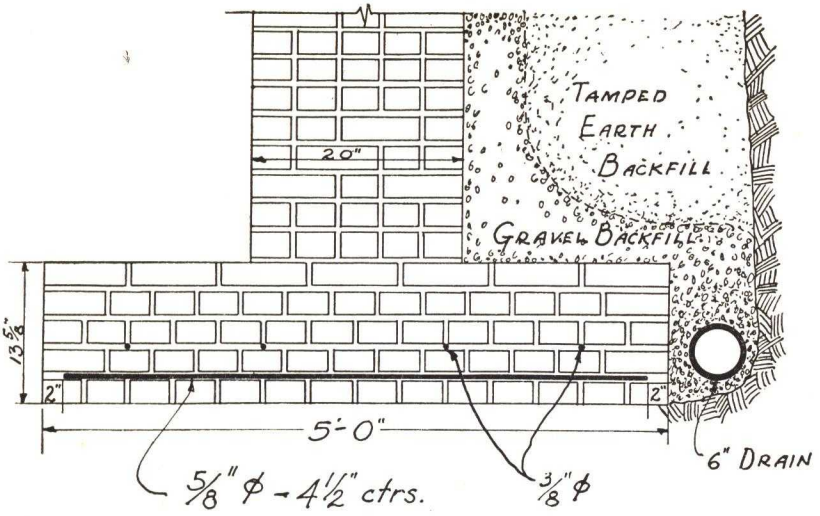
Therefore in our problem, the effective depth required to reduce to the unit shearing stress to within the allowable shear of 35 lbs./sq. inch, should be:

The nearest depth available in normal brick masonry is obtained with four courses, which will be approximately 11 inches. The unit shear will be computed for a depth of 11 inches and investigated on a vertical plant 11″ from face of wall.

$$V = \frac{6000 \times 9}{12} = 4500 \text{ lbs.}$$

$$v = \frac{V}{bjd} = \frac{4500}{12 \times .866 \times 11} = 39.4 \text{ lbs./sq. in.}$$

As was expected, this exceeds the allowable shear of 35 lbs./sq. inch, but the effective depth of 11″ shall be considered sufficient, inasmuch as the compensating weight of the footing and earth over the footing will actually reduce the computed moment and also the shear.




**Bond**

The bond stress on the reinforcement will be investigated on a plane at the face of the wall. The shear on this plane is:

$$V = 6000 \times \frac{20}{12} = 10{,}000 \text{ lbs.}$$

The unit bond stress $u = \frac{v}{bjd} = \frac{10{,}000}{12 \times .866 \times 11} = 87.5$ lbs./sq. in.

The allowable bond stress for plain bars is 90 lbs./sq. inch and therefore, the design is satisfactory.

One protecting course of brickwork below the reinforcing steel will make the footing five courses deep. The mortar joint between the first and second courses of brick will be ⅞" thick to provide sufficient space for the ⅝" round bars. All other joints will be made ½" thick. The total depth of footing is 13⅝".

The reinforcement rods are placed in the first mortar bed and should stop 2" back of the footing edge, making them 4'-8" long.

If some longitudinal reinforcement parallel to the face of the wall is desired, ⅜" round rods may be placed in the second horizontal mortar joint, which is the one immediately above that contacting the transverse reinforcement.

## 1203. COLUMN FOOTINGS

There are several types of column footings. The most common form is the square or rectangular slab supporting a single column. Reinforcing bars are placed in the bottom mortar beds. When the bars are placed perpendicular and parallel to the sides, the arrangement is known as two-way reinforcement. When additional bars are also placed along both diagonals, it is called four-way reinforcement.

The size of a footing is determined by the area necessary to support the total load and is found by dividing the total load by the safe bearing capacity of the soil. The footing is then designed as a cantilever resisting an upward pressure per sq. foot equal to the assumed bearing capacity of the supporting soil; ignoring, on the safe side, the compensating weight of the footing.

The usual condition is a square footing supporting a square column placed symmetrically in the center. If diagonal lines are drawn from the corner of the column to

the corner of the footing, it will be noted that the footing surrounding the column is composed of four equal and similar trapezoids in which a side of the column and the corresponding side of the footing are the parallel sides and the diagonals are the other two sides.

The maximum moment caused by the upward pressure on the trapezoidal area will be in a vertical plane through the short parallel side along the column. The effective depth is determined for the footing directly under the column. The thickness of the footing may be decreased toward the edges without reducing the effective strength. The amount of this reduction in thickness is controlled by the shearing stress as a measure of diagonal tension in a vertical plane parallel to the side of the column and at a distance therefrom equal to the effective depth.

The amount of reinforcing steel calculated to resist the stress due to the moment at the face of the column is placed in both directions parallel to and perpendicular to the sides of the column. It is preferable that the width of each of the two bands of reinforcing steel be not greater than the width of the column. In this two-way manner of reinforcing, it is also desirable to add a few extra rods outside and parallel to each of the reinforcing bands.

If the four-way method of reinforcing a footing is used, the required area of steel, as determined by maximum moment at the column face, is divided by two and that area of steel placed in each of the bands in the four directions; two of the bands being similar to that described for a two-way reinforcement, while the other two bands are parallel to the diagonals of the footing. It is again recommended that an extra rod be placed outside of and parallel to each of the four reinforcing bands.

At least one course of brickwork is placed below the mortar bed containing the reinforcing steel. In reinforced brick masonry footings, it is more convenient to place not more than two bands of reinforcing steel in one mortar bed. In the four-way method of reinforcing it is recommended, therefore, that the diagonal bands be placed one course below the reinforcing bands that are parallel to the side of the footing. The effective depth will be considered as the distance from the top of the footing to the center of the upper bands of steel. In this instance, the overall depth of the footing will be increased by one course of brickwork.

The bond stress is investigated on a vertical plane at the face of the column. If the computed bond stress is greater than the allowable, it will be necessary to hook or bend the ends of the bars. Sometimes the use of deformed bars will provide the additional required bond stress.

## 1204. RETAINING WALLS

A retaining wall is a structure designed to sustain the lateral pressure of some material contained or restrained on one side of the wall. The walls of reservoirs and similar structures containing water or other fluids are frequently designed as retaining walls, but in the usual sense of the term, a retaining wall is associated with earth or other materials possessing more or less frictional stability.

There are two general types of retaining walls, the plain masonry gravity section and the reinforced masonry wall. The former depends upon its massive triangular or trapezoidal section to withstand the lateral thrust and overturning force of the retained material, while the latter utilizes the principles of reinforced masonry to provide a more economical section. The gravity section of masonry retaining wall has enjoyed a long and pleasant history in engineering accomplishments and is naturally used as a guide or criterion in the design of reinforced masonry retaining walls.

The reinforced retaining wall is generally in the shape of an inverted T, in which the base is approximately the same width as the equivalent gravity section and, naturally, the vertical stem is the same height as the altitude of the triangular or trapezoidal

section of a gravity wall. That part of the base or footing of the reinforced retaining wall on the side of the retained material is known as the heel and the extension of the footing on the other side of the main wall or stem is called the toe. The side of the stem or vertical surface of the wall toward the retained material is the back of the wall, while the other side or surface is known as the front or face of the wall.

The problem of actual pressures caused by earth or other partially stable materials cannot be solved with the exactness of mathematics. It is not the intent of this section to be any sort of a treatise on the subject of soil pressures, but rather to indicate a practical method of design. Certain assumptions have to be made regarding the magnitude and direction of such forces. There are several theories regarding earth pressures and, as stated in Chapter 4, the newer science of soils mechanics is providing very valuable information in this field. It is probable that most of the theoretical formulas for earth pressure will be discarded or revised considerably as a result of the data being obtained in this newer approach to the study of the mechanics of soils. Until more definite information is available, many engineers prefer to assume that the earth pressure is similar to that of a fluid.

The equivalent fluid pressure on a gravity wall may be determined, and then the reinforced wall designed so that it will be equally stable. Since the line of pressure and point of application of a fluid is known, the maximum fluid pressure can be determined under which a particular gravity section will remain stable. It will be found that the equivalent fluid weight varies with the relative stability of the various forms or cross-sectional shapes of gravity sections. The type of gravity wall, which is generally considered as corresponding to the reinforced cantilever stem and base, will have different values for the equivalent fluid weight depending upon the relative stability of various ratios of width or thickness (t) of base to height (h) of wall. These values for equivalent fluid weight range from 15 lbs. per cu. ft. for a ratio of t/h equal to $\frac{1}{3}$, to 50 lbs. per cu. ft. for a ratio of t/h equal to approximately $\frac{2}{3}$. The proportions or ratios of t/h most frequently found in reinforced retaining walls vary from 4/10 to 5/10 and the corresponding values for equivalent fluid weights are 20 to 30 lbs. per cu. ft. It is probable that the most general practice in the design of reinforced retaining walls is to use the equivalent pressure of a fluid weighing 30 lbs. per cu. ft. The tables of pressures and moments in the appendix are based on this value.

A reinforced masonry retaining wall must have sufficient stability to resist overturning and sliding. The resistance to overturning is provided by designing the wall so that the resultant pressure passes through the base. When the resultant passes through the base at the extreme edge of the toe, the factor of safety against overturning is unity. It is preferable that the safety factor against overturning be one and a half ($1\frac{1}{2}$) or more. The factor of safety against overturning is the result obtained by dividing the distance from the extreme edge of the toe to that point directly under the center of gravity of the wall by the distance from this latter point to the point where the resultant cuts the base.

The resistance to sliding depends upon the coefficient of friction of the masonry on the supporting soil. The coefficient of friction between masonry and various foundation soils may be assumed as follows: Wet clay, 0.30; sand, 0.40; dry clay, 50; gravel, 0.60. The resistance (due to friction) against sliding is computed by multiplying the vertical pressure by the coefficient of friction. If this value is not greater than the horizontal pressure against the wall, it will be necessary to provide additional resistance by means of a key or steps in the base of footing or some other suitable change in design. The earth in front of a retaining wall increases its stability against sliding, but it is frequently disregarded to provide an additional safety factor.

There is also an infrequent possibility of failure due to crushing of the masonry at the toe and it is important that the maximum unit pressure at the toe does not exceed

the allowable compressive strength of the masonry. This seldom occurs, however, because the maximum pressure per square foot at the toe is rarely more than four or five times the allowable compressive strength of the brick masonry in pounds per square inch.

There is some probability, however, of the maximum pressure at the toe exceeding the supporting value of the soil upon which the wall footing is to rest. In such an event, the wall is redesigned, usually increasing the width of footing.

**Example:**

Design an R-B-M retaining wall to restrain a level earth bank 8′ high with the top of the footing 4′ below grade at the front of the wall, in order to be below the frost line. It is assumed that the allowable strength of the brick masonry ($f_b$) is 700 lbs. per sq. inch; $f_s$ is 16,000 lbs. per sq. inch; the modular ratio, n, equals 15; the weight of the earth is 100 lbs. per cubic foot and the brick masonry as 125 lbs. per cubic foot. The coefficient of friction of the supporting soil is 0.40 and its bearing capacity 2 tons per square foot.

**Solution:**

According to Table No. 81 in the appendix, for n = 15, $f_b$ = 700 and $f_s$ = 16,000, the following other values are given: p = .0087, k = 0.396; j = 0.868 and K = 120.4.

The horizontal pressure on the wall is assumed as equivalent to that of a fluid weighing 30 lbs. per cubic foot. The height of wall above the footing is:

$$h = 8' + 4' = 12'$$

**Thickness of Stem:**

The thickness of the main wall or stem is determined by the maximum moment at the top of footing due to the equivalent fluid pressure.

$$M = wh^3/6 = 30 \times 12^3/6 = 8640 \text{ ft.-lbs.}$$
$$bd^2 = M/K = 8640/120.4 = 71.8$$

since, $b = 1$ foot, $d = \sqrt{71.8} = 8.475''$

The effective depth or thickness, obtained with two 3¾″ wythes or brick with ½″ mortar joint between and half of a 1″ joint for the vertical steel, is 8½″, which will satisfy the above requirement. The total thickness of the main wall will, therefore, be made 12¾″ thick; which will be provided by three wythes of brick, each 3¾″ wide, one mortar joint ½″ wide and the other containing the vertical steel to be 1″ wide.

**Reinforcing Steel in Stem:**

The area of vertical steel required in the stem is:

$$A_s = pbd = .0087 \times 12 \times 8.475 = 0.884 \text{ sq. in.}$$

⅝″ round bars on 4″ centers provides an area of 0.921 sq. in. per ft. and ¾″ round bars on 6″ centers will be exactly 0.884 sq. in. per foot. The ¾″ bars on 6″ centers will be placed vertically in the stem. It is also recommended that some small rods, say ⅜″ round, be placed horizontally in every sixth mortar bed, as indicated.

It will not be necessary to carry all of the vertical reinforcement to the top of the stem or main wall. The bending moment in the wall diminishes to zero at the top. It will be found that at a point 4′-6″ above the top of the footing, the required area of steel is less than half of that computed to resist the moment at the top of footing. At 7′-3″ above the top of footing, or 4′-9″ below top of wall only one-fourth of the bars are required. This can be verified by calculating the moment at those points. It will also be found that the effective depth could be reduced in the same ratio. For instance, at 4′-6″ above top of footing, the required effective depth is less than 4¼ inches. It would

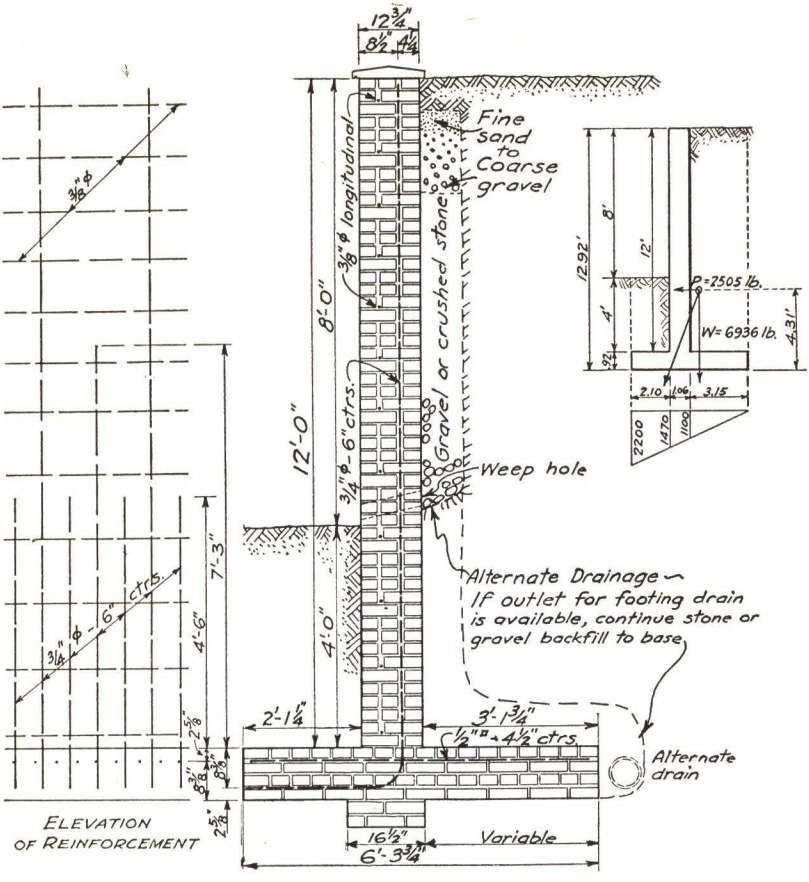



*Figure 32*

be possible, therefore, to reduce the wall thickness to 8½ inches at this point, but most designers prefer to keep the same thickness of wall for the full height. This shall be done in this instance, but every other ¾" round bar in the wall shall be stopped 4'-6" above the footing and alternate bars of those remaining may be stopped at 7'-3" above the footing. In other words, the bars are spaced on 6" centers for a distance of 4'-6" above the footing; thence, the bars are on 12" centers for an additional 2'-9" and on 24" centers for the remaining 4'-9" to top of wall.

**Shear in Stem**

The maximum shear in the main wall occurs at the top of footing and we have:

$$V = wh^2/2 = 30 \times 12^2/2 = 2160 \text{ lbs.}$$

and the unit shear, $v = \dfrac{V}{bjd} = \dfrac{2160}{12 \times 0.87 \times 8.5} = 24.4$ lbs./sq. in.

This is safely within the allowable shearing stress of 40 lbs./sq. inch.

**Bond Stress in Stem**

$$u = \frac{V}{\Sigma ojd} = \frac{2160}{4.71 \times 0.87 \times 8.5} = 88.8 \text{ lbs./sq. in.}$$

This is also within the allowable bond stress for plain bars of 105 lbs. per sq. inch.

**Footing**

The footing will be assumed as 11″ deep (4 courses with bottom and top mortar beds ¾″ thick and middle bed ½″ thick).

**Width of Footing**

Moment at the base of footing due to equivalent fluid pressure:

$$M = wH^3/6 = 30 \times 12.92^3/6 = 10{,}800 \text{ ft.-lb.}$$

Assume that the face of the stem is at the outside edge of the middle third of the base. If the width of the base is indicated as "t", then the toe of the footing or base extends a distance of t/3 in front of the stem. It is desirable to have the resultant of all forces pass through the base of the footing at the outside of the middle third, which in this instance is directly below the face of the stem. Equating moments about the outside of the middle third of the base (t/3 from the toe), we have:

$$4\frac{t}{3}100\frac{t}{6} + 10{,}800 = 0.92t \times 125\frac{t}{6} + 1.06 \times 12 \times 125 \times 0.53 + 12\left(\frac{2t}{3} - 1.06\right)100\left(\frac{t}{3} + 0.53\right)$$

$$400t^2/18 + 10{,}800 = 115t^2/6 + 843 + 800t^2/3 - 674$$

$$267.5t^2 = 10{,}631$$

$$t = 6.31 \text{ ft.} = 6 \text{ ft.-}3\tfrac{3}{4} \text{ in.}$$

Therefore, footing will be made 6′-3¾″ wide.

**Weights**

| | | | | |
|---|---|---|---|---|
| Stem | 1.06 x | 12 x | 125 = | 1590 lbs. |
| Earth Backfill | 3.15 x | 12 x | 100 = | 3780 lbs. |
| Fill over toe | 2.10 x | 4 x | 100 = | 840 lbs. |
| Footing | 0.92 x | 6.31 x | 125 = | 726 lbs. |
| Total vertical load (W) | | | | 6936 lbs. |

As a check on our calculations for the width of the footing, moments will be taken about the toe to locate where the resultant cuts the base.

840 x 1.05 + 1590 x 2.63 + 3780 x 4.74 + 726 x 3.16 — 10,800 = 6936x

x = 2.095, which practically checks 6.31/3 or 2.103.

**Pressure on Foundation**

The average pressure on the foundation is:

$$\frac{W}{A} = \frac{6936}{6.31} = 1100 \text{ lbs. per sq. ft.}$$

Since the resultant intersects the base at the outside edge of middle third, the pressure at the heel is zero, while the pressure at the toe is 1100 x 2 or 2200 lbs. per sq. ft. The maximum unit pressure at the heel is less than the allowable, two tons (4000 lbs.) per sq. foot. This pressure of 2200 lbs. per sq. foot is equal to 2200/144 or 15.3 lbs. per square inch. The allowable compressive strength of the masonry is 700 lbs. per sq. inch and no danger of the masonry crushing.

**Resistance Against Sliding**

The resistance against sliding is obtained by multiplying the coefficient of friction by the vertical load. In this case, the weight of earth in front of the wall will not be considered, inasmuch that fill may not be made as soon as the fill back of the wall. In other computations, however, such as for overturning or maximum pressure on foundation, the fill in front of the wall has been considered, because by so doing, the factor of safety was increased.

Resistance to sliding = 6096 x 0.4 = 2438.4 lbs.

The total horizontal pressure = $wH^2/2$ = 30 x $12.92^2/2$ = 2505 lbs. It will be necessary, therefore, to provide a key 5 or 6″ deep in the base of the footing. It will also be important to specify that the fill be made in front of the wall as the fill in back of the wall is carried up and both thoroughly rolled in layers of not more than 8″ thick. This will provide an additional safeguard against sliding.

**Design of Toe Slab**

In computing the moment in this cantilever, the upward pressure of the supporting soil will be considered and the compensating weight of the footing and backfill will be ignored.

$$M = 1470 \times 2.10 \times 1.05 + 730/2 \times 2.10 \times 2/3 \times 2.10 = 4315 \text{ ft.-lb.}$$

$$bd^2 = \frac{M}{K} = \frac{4315}{120.4} = 35.8$$

Since, (b = 1), d = 5.9″

The effective depth available is $2\frac{1}{4} + \frac{3}{4} + 2\frac{1}{4} + \frac{1}{2} + 2\frac{1}{4} + \frac{3}{8} = 8\frac{3}{8}$″

**Reinforcing Steel in Toe Slab**

$A_s$ = pbd = .0087 x 12 x 5.9 = 0.616 sq. in. $\frac{5}{8}$″ round bars on 6″ centers will provide approximately the required area, but it is probable that the $\frac{3}{4}$″ round bars on 6″ centers in the stem will be bent into the footing.

**Shear in Toe**

$$V = \frac{1470 + 2200}{2} \times 2.10\text{-}2.10 \times 0.92 \times 125 = 3608 \text{ lbs.}$$

$$\text{Unit shear, } v = \frac{V}{bjd} = \frac{3608}{12 \times 0.87 \times 8.38} = 41.2 \text{ lbs.}$$

The allowable shear is 40 lbs. per sq. inch if no special anchorage is made. Inasmuch as the computed shearing stress is more than the allowable, it appears that stirrups may be required or the bars bent at the ends to obtain the increased allowable shearing stress of 60 lbs. per sq. in. However, the excess of 1.2 lbs. per sq. inch is so small that most designers would ignore it; particularly due to the fact that the weight of the earth in front of the wall will help to reduce the shearing stress by 9.6 lbs. per sq. inch.

**Bond Stress in Toe**

$$\text{Unit bond stress, } u = \frac{V}{\Sigma ojd} = \frac{1593}{4.71 \times 0.87 \times 8\frac{3}{8}} = 46.5 \text{ lbs./sq. in.}$$

The allowable bond stress is 105 lbs. per sq. in.

**Design of Heel Slab**

The heel is also designed as a cantilever slab and the maximum moment is, of course, at the support, which is the back of the stem. There is the upward pressure of the supporting soil and the downward load of the earth backfill, as well as the weight of the heel slab itself.

$$M = 3780 \times 3.15/2 + 362 \times 3.15/2 - 1100 \times 3.15/2 \times 3.15/3$$
$$M = 5950 + 570 - 1820 = 4700 \text{ ft.-lb.}$$
$$bd^2 = \frac{M}{K} = \frac{4700}{120.4} = 39$$

Since b = 1 foot, $d^2 = 39$, and
d = 6.25 in.

The effective depth available is actually 8⅜".

**Reinforcing Steel in Heel**

$A_s = pbd = .0087 \times 12 \times 6.25 = 0.653$ sq. in. per lin. ft.

½" square bars on 4½" centers equals 0.667 sq. in/ft.

⅝" round bars on 5½" centers provides 0.67 sq. in./ft.

¾" round bars on 8" centers would provide an area of 0.663 sq. in. per foot, which is slightly under, but for all practical purposes would be acceptable. It is probable, however, that either the ½" square or ⅝" round bars would be preferred by most designers. We shall use the ½" square bars on 4½" centers because of a slight aid in construction; i.e., the vertical bars in the stem are to be on 6" centers and therefore, every fourth bar in the heel will be placed alongside every third vertical bar.

**Shear in Heel**

$$V = 3780 + 362 - 1100 \times 3.15/2 = 2410 \text{ lbs.}$$
$$\text{Unit shear, } v = \frac{V}{bjd} = \frac{2410}{12 \times 0.87 \times 8.38} = 27.6 \text{ lbs./sq. in.}$$

This is safely within the allowable shearing stress of 40 lbs. per sq. inch and no stirrups or special anchorage will be required.

**Bond in Heel**

$$\text{Unit bond stress, } u = \frac{V}{\Sigma ojd} = \frac{2410}{5.35 \times 0.87 \times 8.38} = 61.9 \text{ lbs./sq. in.}$$

This is safely within the allowable bond stress of 105 lbs. per sq. inch for plain bars.

## 1205. COUNTERFORTED RETAINING WALLS

In the counterfort type of retaining walls, the main walls or stem is supported at intervals by vertical ribs (counterforts) along the back of the wall; although in some designs, counterforts are also placed along the front of the wall as well as the back. The experience of engineers in reinforced concrete construction has been that this type of retaining wall is more economical only when the height is greater than 20 feet. Since R-B-M walls do not require the bothersome forms used in concrete construction, it is possible that in reinforced brick masonry the counterfort type of retaining wall may be worthy of consideration for heights less than 20 feet.

The vertical slab (stem) in the counterfort wall, instead of acting as a cantilever from the footing, is assumed and designed as composed of horizontal elements or beams continuous over the supports afforded by the counterforts. The maximum bending moment in the intermediate slabs is assumed as equal to $\frac{wl^2}{12}$ and that for the end slabs at corners as $\frac{wl^2}{10}$. In the design of these horizontal beams, consideration must be given to the increase of equivalent fluid pressure towards the bottom. The wall slab is usually designed so that changes in the thickness of wall and spacing of steel are made at a comparatively few points to simplify construction.

The heel of the footing (back of wall) is also designed as a continuous slab between counterforts, instead of being a cantilever. However, the toe of the footing is designed as a cantilever supported at the face of the wall, unless there are also counterforts in front of the wall. This latter feature is rare.

The counterforts are designed as cantilever beams fixed at the footing and diminishing in effective depth (measured horizontally) towards the top. The pressure against the main wall tends to push that wall away from the counterforts and it is necessary, therefore, to tie the counterfort to both the main wall and footing by means of the reinforcement steel.

The main reinforcement in the cantilever counterfort is near and parallel to the back or edge towards the fill. This reinforcement is anchored in the footing and in the top of the wall. Horizontal ties are placed in the counterfort at every sixth or eighth course and the ends bent, at the front around the wall reinforcement and at the back around the main counterfort reinforcement.

## 1206. CONSTRUCTION OF RETAINING WALLS

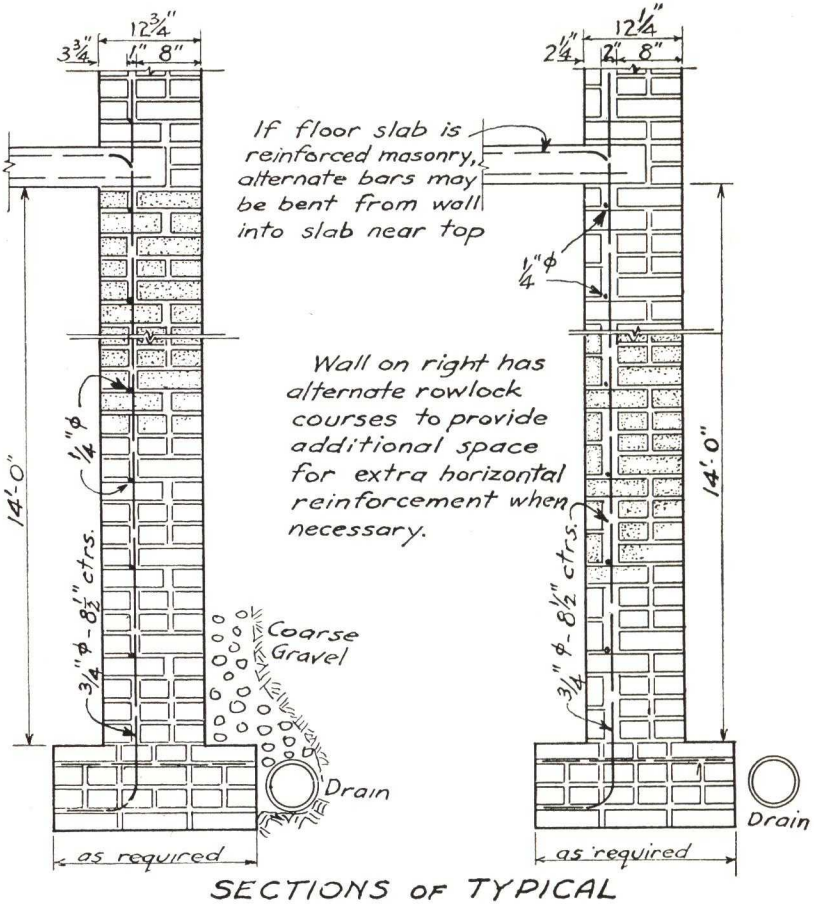
Good construction is every bit as essential as good design. The materials and workmanship in the walls is important, but too much emphasis cannot be placed on the importance of adequate drainage and manner of placing the backfill. Coarse gravel or crushed stone should be placed against the back of the wall from the weep holes to within two feet of the surface. A finer grading to sand should be made in the last 18 inches.

The weep holes are made with small (3″ or 4″) vitrified clay tile placed in the wall a few inches above the footing and sloping to the front. These tile are placed in the wall approximately 20 feet apart and may connect to a 6″ drain laid parallel to the back of the wall at the bottom of the gravel backfill, or merely extend through the wall with a piece of copper wire mesh covering the opening to keep the gravel out of the tile. When the 6″ drain tile parallel to the back of the wall is used, it is usually laid level on a firmly tamped base with unfilled joints wrapped in strips of copper wire mesh. This tile drain is not recommended with counterforted walls because of the piecemeal construction.

The earth fill in back of the wall, as well as that in front, should be placed in layers not more than 8 inches thick and consolidated with the earth below by rolling with "sheep's foot" rollers or other approved methods. Hand tamping is necessary at the wall and around counterforts. Wherever possible, it is desirable to place the earth fill in front of the wall up to its required grade at approximately equal stages of elevation with the backfill.

## 1207. R-B-M BASEMENT WALLS

The use of reinforced brick masonry for basement foundation walls results in an economical section of wall which is capable of resisting relatively high tensile stress due to soil pressures. While many engineers and architects design reinforced basement foundations as retaining walls, considerable saving is effected by analyzing most walls as outlined in Section 408. As a general thing, a reinforced foundation wall with dowels or bars extending into the footing may be considered as fixed at the footing. If the reinforcement extends into the masonry walls above the first floor elevation, the assumption is also made that the wall is continuous at the first floor support. If lateral supports, afforded by piers, cross-walls and end walls, are spaced not more than twice the height of the wall, it is worth-while analyzing the wall as a vertical slab supported along four edges, with load distributed in both the vertical and horizontal directions.




SECTIONS OF TYPICAL
R-B-M FOUNDATION WALL

# CHAPTER 13

# BUILDING REGULATIONS FOR REINFORCED BRICK MASONRY

The following building regulations are presented in the form of building codes. However, sections relating to materials and tests, construction practices and equipment, may be used as specifications for reinforced brick masonry.

## SECTION 1 — GENERAL

### 13-101. SCOPE

(a) The following rules and regulations pertain to the design and construction of reinforced brick masonry in any structure to be erected under the provisions of this building code of which they form a part.

### 13-102. PERMITS AND DRAWINGS

(a) Drawings and typical details of all reinforced brick masonry construction showing the sizes and position of all structural members, steel reinforcement, design strengths, and the live load used in the design, shall be filed with the building department before a permit to construct such work shall be issued. Calculations pertaining to the design shall be submitted, if so ordered by the Commissioner of Buildings.

### 13-103. TERMS, SYMBOLS AND NOTATIONS

(a) The following terms are defined for use in this code:

**Abutment** — That part of a structure which takes the thrust of a beam, arch, vault, truss or girder.

**Anchor** — A piece or connected pieces of metal used for tying together two or more pieces of masonry materials.

**Bar** — Steel reinforcement rod.

**Base Course** — The lowest course of masonry in a wall or pier. A footing course. Frequently, a belt or trim course at the grade line is termed a base course.

**Bat** — A piece of broken brick.

**Bed** — The prepared layer of mortar on or in which a piece of masonry material is laid.

**Bond** — The tying of the various pieces and parts of a masonry wall, by laying one piece across two or more pieces; the entire system of bonding or breaking joints as used in masonry construction. The mortar between bricks is sometimes termed a bond, but the more general use refers to the adhesion between mortar and masonry units or the adhesion between mortar and reinforcement.

**Brick** — A structural or road building unit of clay or shale, formed while plastic into a rectangular prism, solid or practically so, and hardened by heat.

**Brick Masonry** (Brickwork) — Any structure or structural part, made of bricks and mortar.

**Buttering** — Placing mortar on a brick with a trowel before brick is laid.

**Center** — A timber framework or mold upon which the masonry of an arch, vault or beam is supported until self-supporting.

**Closer** — The last brick laid in a course; the end bricks of a part of a course, fitted at the openings. A closer may be a whole brick or less in size.

**Column** — An upright support, the length of which exceeds three times its least lateral dimension.

**Combination Column** — A column in which a structural steel section, designed to carry the principal part of the load, is encased in brick masonry of such quality that some additional load may be allowed.

**Concrete** — A mixture of Portland cement, fine aggregate, coarse aggregate and water.

**Coping** — The material or number used to form a capping or finish on top of a wall, pier, etc., to protect the masonry below by throwing off the water to one or more sides.

**Corbel** — That part of the masonry built outward from the face of masonry by projecting successive courses of the masonry.

**Course** — One of the continuous horizontal layers of masonry units which are bonded together and thus form a masonry structure.

**Deformed Bar** — Reinforcing bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling so as to firmly engage the surrounding mortar or concrete. Wire mesh with welded intersections not farther apart than twelve inches in the direction of the principal reinforcement and with cross wires not smaller than No. 10 may be rated as a deformed bar.

**Effective Area of Brick Masonry** — The area of a section which lies between the centroid of the tensile reinforcement and the compression face of the structural member.

**Effective Area of Reinforcement** — The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and that for which the effectiveness of the reinforcement is to be determined.

**Face** — The front or exposed surface of a wall.

**Footing** — That part of a foundation wall resting on the bearing soil, rock, piling, etc., which transmits the superimposed load to the bearing material.

**Foundation** — That part of a load-bearing wall, below the level of the adjacent grade, or below the level of the first tier of floor beams or joists.

**Grout** — A mixture of cementitious material (cement, lime), sand and sufficient water to make a consistency that will flow without separation of ingredients.

**Header** — A brick laid lengthwise across a wall and serving as a bond.

**Lintel** — A beam of suitable material placed over a wall opening to carry the superimposed weight above.

**Mortar** — A plastic mixture of cementitious material, sand and water.

**Pointing** — Finishing a mortar joint with a tool or trowel after a brick is laid.

**R-B-M** — Abbreviation for reinforced brick masonry.

**Reinforced Brick Masonry** (R-B-M) — Brick masonry in which metal is imbedded in such a manner that the two materials act together in resisting forces.

**Reinforcement** — Structural Steel Shapes, steel bars, rods, wire mesh or expanded metal imbedded or encased in brick masonry to increase the strength of the masonry.

**Rods** — Steel reinforcement bars.

(b) The symbols and notations are similar to those used in reinforced concrete design, as follows:

$a$ = Width of face of column or pedestal.

$A$ = Total area of top of pedestal, pier, or footing at the column base.

$A'$ = Loaded area of pedestal, pier, or footing at the column base.

$A_c$ = Area of core of a reinforced column measured to the outside of the lateral ties.

$A_b$ = Net area of brick masonry section of a combination column.

$A_g$ = The overall or gross area of reinforced columns; the total area of the brick masonry encasement of combination columns.

$A_r$ = Area of the steel core in a combination column.

$A_s$ = Effective cross-sectional area of reinforcement in tension in beams or in compression in columns.

$A_v$ = Total area of web reinforcement in tension within a distance, s, (measured perpendicular to the direction of the web reinforcement bar), or the total area of all bars bent up in any one plane.

b = Width of rectangular beam or width of flange of T-beam.

b′ = Thickness of web in T-beams.

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement (effective depth); the least lateral dimension of an R-B-M column.

d′ = Total depth of a horizontal member.

D = Deflection of a horizontal member.

e = Eccentricity of the resultant load on a column, measured from the gravity axis.

$E_b$ = Modulus of elasticity of brick masonry in compression.

$E_s$ = Modulus of elasticity of steel in tension or compression (30,000,000 lbs. per sq. in.).

$f_b$ = Compressive unit stress in extreme fibre of brick masonry in flexure or axial compression in brick masonry in columns.

$f'_b$ = Ultimate compressive strength of brick masonry as determined by test specimens.

$f_r$ = Permissible unit stress in the metal core of a combination column.

$f'_r$ = Permissible unit stress on unencased steel columns.

$f_s$ = Permissible unit stress in longitudinal reinforcement.

$f'_s$ = Useful limit stress or yield point of lateral reinforcement in columns.

$f_v$ = Tensile unit stress in web reinforcement.

h = Unsupported length of column.

I = Moment of inertia of a section about the neutral axis for bending.

j = Ratio of lever arm between centroid of compression and centroid of tension to the depth (d).

k = Ratio of depth of neutral axis (from compressive face of slab or beam) to effective depth, d.

K = Coefficient of resistance of the steel ($f_spj$) or brickwork ($1/2f_bkj$).

$l$ = Span length of beam or slab.

$l'$ = Clear span for positive moment and the average of the two adjacent clear spans for negative moment.

M = Bending moment or moment of resistance in general.

n = Ratio of modulus of elasticity of steel to that of brick masonry = $E_s/E_b$.

$\Sigma o$ = Sum of perimeters of bars in one set.

p = Ratio of effective area of tensile reinforcement to effective area of brickwork in beams.

$P_g$ = Ratio of the effective cross-sectional area of vertical reinforcement to the gross area $A_g$ of columns.

p′ = Ratio of volume of lateral reinforcement to the volume of the brick masonry core (out to out of lateral ties) of a reinforced column.

P = Total allowable axial load on a column whose length does not exceed 10 times its least cross-sectional dimension.

P′ = Total allowable axial load on a long column. (Slenderness ratio 10 to 20.)

$P_b$ = Permissible unit working stress in brickwork on top of a pedestal, pier, or footing directly under a column.

r = Least radius of gyration of a section.

R = Ratio of gross area to core area of a reinforced brick masonry column, $A_g/A'_c$.

s = Spacing of stirrups or of bent bars in a direction parallel to that of the main reinforcement.

t = Thickness of the flange of T-beams.

u = Bond stress per unit of surface area of bar.

v = Shearing unit stress.

V = Total shear.

V′ = Excess of the total shear over that permitted on the brick masonry.

w = Uniformly distributed load per unit of length of beam or slab.

w′ = Uniformly distributed dead and live load per unit of area of a floor or roof.

W = Total dead and live load uniformly distributed over a single panel area.

## SECTION 2 — MATERIALS AND TESTS

### 13-201. TESTS

(a) The Commissioner of Buildings, or his authorized representative shall have the right to order the test of any material entering into reinforced brick masonry, when there is reasonable doubt as to its suitability for the purpose; to order reasonable tests of the brick masonry from time to time to determine whether the materials and workmanship are such as to produce R-B-M of the necessary quality; and to order the test under load of any portion of a completed structure, when the conditions have been such as to leave reasonable doubt as to the adequacy of the structure to serve the purpose for which it is intended.

(b) Tests of brick and other materials shall be made in accordance with the requirements of the American Society for Testing Materials. The complete records of such tests shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work, and shall be preserved by the engineer or architect for two years after the completion of the structure.

### 13-202. LOAD TESTS

(a) When a load test is required, the member or portion of the structure under consideration shall be subject to a superimposed load equal to one and one-half times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of the structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or, where lawful, a lower rating shall be established. The structure shall be considered to have passed the test if the maximum deflection at the end of the twenty-four hour period does not exceed the value of D as given by the following:

$$D = \frac{.001\ l^2}{12d'}$$

in which, $l$ is the span, d′ is the total depth of the slab or beam and D is the maximum deflection — all expressed in the same units.

If the deflection exceeds the value of D as given in the above formula, the construction shall be considered to have passed the test if within twenty-four hours after the removal of the load the slabs or beams show a recovery of at least seventy-five per cent of the observed deflection.

### 13-203. SUPERVISION

(a) All reinforced brick masonry shall be supervised by the architect or engineer responsible for its design, or by a competent representative responsible to the architect or engineer. A record shall be kept of such supervision, which shall cover the quality,

quantity and placing of materials. A complete record shall also be kept of the progress of the work and when the temperature falls below 40 degrees F., a record of temperatures and of the protection while curing. This record shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work and shall be preserved by the architect or engineer for two years after the completion of the work.

### 13-204. BRICK

All brick shall be sound, whole, new clay or shale brick. Brick used for parapet walls, garden walls, or in masonry below grade or in contact with earth, such as foundations, retaining walls, walks, terraces, etc., shall conform to the Standard Specifications for Building Brick, Grade SW, A.S.T.M. C62-37T.

Brick used as facing for exterior walls, exposed to the weather on one side only and not in contact with earth, shall conform to Standard Specifications for Building Brick, Grade MW, A.S.T.M. C62-37T. Brick used as back-up in walls not in contact with earth or in interior walls or partitions, shall conform to Standard Specifications Grade NW, A.S.T.M. C62-37T.

NOTE: The above recommendations are, in general, applicable to exposures encountered in the Northeastern quarter of the United States. In regions characterized by less severe frost action or by dryer climate than is found in the Northeastern quarter of the United States, Grade MW brick may be specified in lieu of Grade SW.

### 13-205. PORTLAND CEMENT

Portland cement shall conform to the current A.S.T.M. Specifications for Portland Cement, C9-38.

### 13-206. LIME

Lime shall conform to the A.S.T.M. Specifications (C5-26) for Quicklime or (C6-31) for Hydrated Lime for Structural Purposes.

### 13-207. LIME PUTTY

The slaking of quicklime and preparation of lime putty shall conform to the suggestions contained in the appendix to the A.S.T.M. Specifications for Quicklime for Structural Purposes (C5-26). The lime putty shall be soaked for a sufficient period to produce a highly plastic lime putty.

### 13-208. SAND

All sand used for mortar or grout shall be clean, hard, well-graded, free from organic matter or other objectionable impurities and containing not more than 2% of loam. It shall all pass a No. 8 sieve and have a fineness modulus not less than 1.90 and not more than 2.50.

### 13-209. WATER

Water used in mixing mortar shall be clean, and free from deleterious amounts of acids, alkalis, or organic materials.

### 13-210. MORTAR

The mortar used in R-B-M shall be highly plastic and comply with Section 13-303. A recommended mix consists of one part Portland cement, 0.20 part lime putty and three parts sand by weight. (The equivalent volume proportions are approximately 1:0.22:3½.)

### 13-211. GROUT

The grout used in R-B-M shall be composed of the same original proportions given for mortar, to which is added the minimum amount of water necessary to produce a consistency which will permit the grout to flow freely.

### 13-212. REINFORCEMENT

(a) Metal reinforcement shall conform to the requirements of the A.S.T.M. Specifications for Billet-Steel, A15-35 or Rail-Steel Reinforcement Bars, A16-35.

(b) Welded wire fabric or cold-drawn wire for reinforcement shall conform to the requirements of the A.S.T.M. Specifications for Cold-Drawn Steel Wire for Reinforcement, A82-34.

(c) Structural steel shall conform to the requirements of the current A.S.T.M. Specifications for Structural Steel for Buildings, A9-36.

### 13-213. STORAGE OF MATERIALS

(a) Cement, lime and sand shall be stored in such a manner as to prevent deterioration or intrusion of foreign matter. Any material which has become unsuitable for good construction shall not be used in R-B-M.

## SECTION 3

## WORKING STRESSES AND TESTS FOR R-B-M

### 13-301. ULTIMATE STRENGTH OF BRICK MASONRY

For the design of reinforced brick masonry structures, the value of $f'_b$ used for determining the working stresses stipulated in this chapter shall be based on the specified minimum ultimate 60-day compressive strength of the brickwork or on the specified minimum ultimate compressive strength at the earlier age at which the R-B-M may be expected to receive its full load. All plans, submitted for approval or used on the job shall clearly show the assumed strength of brick masonry at a specified age for which all parts of the structure were designed.

### 13-302. TESTS ON BRICK MASONRY

(a) The Commissioner of Buildings may at his discretion require a reasonable number of compression tests to be made during the progress of the work. Such tests shall be made on prisms built of the same materials with the same workmanship and in so far as possible, the same bonding arrangement that is used in the structure. Unless permission is otherwise given, all test specimens shall have a height to thickness ratio (h/d) not less than 3. All ultimate compressive strengths ($f'_b$) shall be expressed as the equivalent strength of a specimen having a h/d ratio of 6. This value may be obtained by multiplying the compressive strength of a specimen having a h/d ratio less than 6 by the correction factor indicated in the following table:

**Correction Factors for Ultimate Strength of Brick Masonry.**

| Ratio of height to (h/d) Thickness or Depth..... | 1.5 | 2.0 | 2.5 | 3.0 | 4.0 | 5.0 | 6.0 |
|---|---|---|---|---|---|---|---|
| Correction Factor..... | 0.59 | 0.67 | 0.75 | 0.80 | 0.89 | 0.96 | 1.00 |

(b) The prisms for beams and slabs are approximately 8 inches square by 25 inches long, built in horizontal position with unselected brick laid as stretchers in running bond, two brick wide, approximately 25″ long and three courses high with ½″ joints. The Commissioner may require that the middle course of such specimens be laid in Flemish or all header bond. The brick are laid flat on a smooth surface, with the outside edge of the ends buttered. The extreme ends of the longitudinal vertical joint are filled with mortar by buttering the inside edge at the ends of the first and last brick. The inside vertical joints are then filled with grout.

A flat bed of mortar is spread for the second course and the brick, mortar and grout placed as described for the first course. The third course is similar to the second. The mortar joints are tooled slightly concave.

For walls, test specimens 8″ x 16″ x 24″ high shall be constructed in the same manner and with the same type of bonding that will be used in the structure.

For columns, test specimens 8″ x 8″ x 24″ high shall be constructed in so far as possible with the same bonding arrangement that will be used in the column.

(c) The prisms are cured in air at a uniform temperature of 70 degrees Fahrenheit until tested.

(d) At the end of 60 days, the prisms are capped and tested on end. The average compressive strength for not less than three prisms shall be used to determine the value for ($f'_b$). Tests may be made at 7 and 28 days, provided that the relation between 7, 28 and 60 day strengths is established by actual tests of representative specimens.

(e) If the ultimate strength ($f'_b$), calculated from a set of prisms, falls below the required strength, the Commissioner of Buildings shall have the right to order a change in the quality of materials used and may require that load tests in accordance with Section 202 be made on that part of the construction represented by said prisms.

### 13-303. TESTS ON MORTAR

(a) All mortar used in R-B-M construction shall be tested in accordance with the Tentative Method of Test for Compressive Strength of Portland Cement Mortars, A.S.T.M. Designation C109-37T, and shall develop the compressive strengths indicated in the following table:

| Specimen | 7 days | 28 days |
|---|---|---|
| 2″ cube | 1500 p.s.i. | 2000 p.s.i. |

(b) At least three specimens shall be made for each lot of 50,000 brick or less.

(c) Special tests of grout other than that used in the brick prisms, are not required, unless specifically requested by the Commissioner of Buildings. The mortar specimens will usually be accepted as an indication of the quality of grout made from that mortar.

### 13-304. ALLOWABLE UNIT STRESSES IN REINFORCEMENT

(a) The unit stresses ($f_s$) in reinforcing steel shall be similar to that allowed for reinforced concrete or structural design. In no event shall the allowable unit stresses exceed the following:

**In Tension**

Reinforcing bars (intermediate grade billet steel or rail steel) ..... $f_s$ = 20,000 lbs. per sq. in.

Web reinforcement ..... $f_s$ = 20,000 lbs. per sq. in.

## 13-305. ALLOWABLE UNIT STRESSES IN R-B-M

The unit stresses in pounds per square inch for brick masonry to be used in the design shall not exceed the values shown in the following table where $f'_b$ equals the average ultimate compressive strength of test prisms at 60 days.

| Type of Stress | Symbol | Approximate % of $f'_b$ | Allowable Unit Stresses in Brick Masonry, when Ultimate Strength ($f'_b$) as determined and controlled by Test Prisms amounts to: | | | | | |
|---|---|---|---|---|---|---|---|---|
| Ultimate Compressive Strength.... | $f'_b$ | ..... | 1200 | 1500 | 1800 | 2100 | 2400 | 3000 |
| **Flexure:** | | | | | | | | |
| Extreme fiber stress in compression | $f'_b$ | 33⅓ | 400 | 500 | 600 | 700 | 800 | 1000 |
| Extreme fiber stress in compression adjacent to supports of continuous or fixed beams or of rigid frames... | $f_b$ | 40 | 500 | 600 | 700 | 800 | 950 | 1200 |
| **Bearing: (1)** | | | | | | | | |
| On full area..... | $f_b$ | 25 | 300 | 375 | 450 | 525 | 600 | 750 |
| On one-third area..... | $f_b$ | 33⅓ | 400 | 500 | 600 | 700 | 800 | 1000 |
| Pedestals (See Chapter 11) | | | | | | | | |
| **Shear:** | | | | | | | | |
| Beams with no web reinforcement and without special anchorage of longitudinal steel..... | v | 2 | 25 | 30 | 35 | 40 | 50 | 60 |
| Beams with no web reinforcement but with special anchorage of longitudinal steel..... | v | 3 | 35 | 45 | 55 | 60 | 75 | 90 |
| Beams with properly designed web reinforcement but without special anchorage of longitudinal steel... | v | 4 | 50 | 60 | 70 | 80 | 100 | 120 |
| Beams with properly designed web reinforcement and with special anchorage of longitudinal steel... | v | 6 | 70 | 90 | 110 | 120 | 150 | 180 |
| Footings where longitudinal bars have no special anchorage..... | v | 2 | 25 | 30 | 35 | 40 | 50 | 60 |
| Footings where longitudinal bars have special anchorage..... | v | 3 | 35 | 45 | 55 | 60 | 75 | 90 |
| **Bond: (2)** | | | | | | | | |
| In beams and slabs and one-way footings: | | | | | | | | |
| Plain bars or structural shapes.. | u | 5 | 60 | 75 | 90 | 105 | 120 | 150 |
| Deformed bars..... | u | 6 | 75 | 90 | 105 | 125 | 140 | 175 |
| In two-way footings: | | | | | | | | |
| Plain bars or structural shapes. | u | 3 | 35 | 45 | 50 | 60 | 70 | 90 |
| Deformed bars..... | u | 3¾ | 45 | 55 | 65 | 80 | 90 | 110 |
| Modular Ratio ($E_s/E_b$)..... | n | ..... | 25 | 20 | 18 | 15 | 15 | 12 |

(1) The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

(2) Where special anchorage is provided, one and one-half times these values in bond may be used.

**13-304 (Continued)**

Structural steel shapes . . . . . $f_s$ = 18,000 lbs. per sq. in.

Welded wire fabric or other steel reinforcement, not exceeding ¼" in diameter, fifty per cent of the minimum yield point as established by A.S.T.M. Standards for that particular grade of steel, but not to exceed . . . . . $f_s$ = 30,000 lbs. per sq. in.

**In Compression**

Intermediate grade steel bars . . . . . $f_s$ = 16,000 lbs. per sq. in.

Rail or hard grade steel bars . . . . . $f_s$ = 18,000 lbs. per sq. in.

Structural steel section in combination columns . . . . . $f_s$ = 18,000 lbs. per sq. in.

## SECTION 4 — CENTERING AND FORMS

### 13-401. DESIGN OF FORMS

(a) All centering shall be thoroughly braced and of adequate strength and rigidity to carry the loads which may be placed upon it during construction without settlement or movement.

**(b) Soffit Boards (For Girders and Beams)**

Soffit boards shall be of such thickness and shall be supported at such intervals as will prevent undue deflection under the weight of the fresh masonry. The soffit board shall be of approximately the same width as the member it supports and shall in no case be of a narrower width than that required to support the outside wythes of bricks without tipping. Soffit boards may be of dressed or rough lumber. They shall be straight and free from twists or such other defects as would prevent obtaining the desired surface.

**(c) Slab Forms**

Forms for slabs may be of square edge boards, either rough or dressed. Where form boards do not abut tightly, the openings shall be filled in or covered so as to prevent the escape of mortar. In lieu of board forms for slabs, the contractor may use sheet metal, Masonite, Plywood or other suitable covering provided that such covering possess suitable rigidity and tightness and is supported at such intervals as will prevent undue deflection between the supporting shores during construction.

### 13-402. CONDITION OF FORMS

All forms, except those made of steel or other impervious materials shall, except in freezing weather, be thoroughly wetted or oiled before any bricks are laid. All surfaces on which bricks are to be laid shall be free from set mortar, wood chips and other dirt or debris.

### 13-403. REMOVAL OF SHORES AND FORMS

(a) In no case shall shores and forms be removed until it is certain that the masonry has hardened sufficiently to carry its own weight and all other reasonable temporary loads that may be placed on it during construction. Before any shores or forms are removed, the masonry shall give a clear ringing sound when struck with a hammer.

(b) For girders and beams, the minimum time which shall elapse before removal of shores or forms is usually ten (10) days after the completion of the member provided that suitable curing conditions have obtained during that period. The forms and shores

under the slabs are usually not removed in less than seven (7) days after completion of such slabs and then only when suitable curing conditions have obtained throughout the entire curing period.

## SECTION 5 — REINFORCING STEEL

### 13-501. RUST, Etc.

*Before being placed, all bars shall be free from loose mill or rust scale, grease, oil, clay or other foreign substances that would reduce or destroy the bond.*

### 13-502. BENDING

All steel reinforcement shall be accurately cut to length and cold bent by such methods as will prevent injury to the material. All kinks or bends in bars caused by handling incident to delivery shall be carefully straightened out without injury to the metal before being placed in the masonry. All bars shall be accurately bent to the dimensions indicated on the shop drawings.

### 13-503. SPACING OF BARS

The minimum center to center spacing between parallel bars shall be one and one-half (1½) times the maximum diameter for round bars, or two (2) times the side dimensions for square bars.

### 13-504. SPLICING

Splices may be made only at such points, and in such manner, that the structural strength of the member will not be reduced. Splices shall provide sufficient lap to transfer the stress between the bars by bond and shear.

### 13-505. SPECIAL ANCHORAGE OF BARS

When required, bars shall be specially anchored. Special anchorage shall be obtained by one of the following methods:

1. By carrying the bottom reinforcement approximately fourteen (14) diameters beyond the face of the supports.

2. By bending the reinforcement around other bars or steel shapes so placed that they will resist longitudinal movement of the steel to be anchored.

3. By bending the bar ends into a semi-circular or square hook of such design as will insure proper bearing on the masonry.

4. In case the ends of a beam or slab are continuous, anchorage shall be obtained by carrying the top reinforcement approximately fourteen (14) diameters beyond the one-fifth (1/5) point of the adjoining clear span.

### 13-506. PROTECTION OF BARS

All bars shall be completely embedded in mortar. All reinforcing steel shall have a coverage of masonry not less than the following:

(a) Three inches (3″) for bottoms of footings.

(b) Two inches (2″) on vertical members where masonry is exposed to action of weather or soil.

(c) One and one-half inches (1½″) for all reinforcement in columns.

(d) One and one-half inches (1½″) on the bottom and sides of beams or girders.

(e) Three-fourths inch (¾″) on bottoms of all floor slabs except where the underside of such slabs is plastered with not less than one-half inch (½″) of cement or gypsum plaster; in which case the protection of the bars may be reduced to one-half inch (½″).

(f) Three-fourths inch (¾″) from the faces of all walls not exposed to action of weather or soil.

(g) One (1) bar diameter over all bars at the upper faces on any member except where exposed to weather or soil, in which case the minimum coverage shall be two inches (2″).

### 13-507. LOCATION AND SUPPORT OF BARS

Reinforcing steel shall be accurately located in the masonry as shown on the design drawings. All horizontal reinforcement in girders, beams, walls and columns shall be supported by a suitable number of wire nails or short pieces of galvanized wire of proper size placed at proper intervals, but in no case shall the spacing exceed four (4) feet. The size of wire nails or galvanized wire which shall be used shall be such as will prevent the reinforcing steel from coming into direct contact with the surface of the bricks.

## SECTION 6 — BRICK MASONRY CONSTRUCTION

### 13-601. WETTING BRICKS

All brick before being laid, except in freezing weather, shall be wetted in the stock pile or elsewhere as may be suitable, for such a period as may be necessary to supply the brick with sufficient moisture to effect proper bond between the brick and mortar. The rate of absorption of a brick may be determined by submerging the flat face (30 sq. in.) one-eighth (⅛) inch in distilled water for one minute and recording the increase in weight in grams. For brick in contact with mortar, a rate of absorption of approximately 20 grams per minute is desirable; while brick in grouted masonry may have a rate of absorption of approximately 30 grams per minute.

### 13-602. MIXING MORTAR OR GROUT

(a) All mixing of mortar and grout shall preferably be done in a mechanically operated batch mixer of the drum type. The use of a continuous mortar mixer will not be permitted. Hand mixing will be permitted on small jobs provided that the quantity of materials and water is controlled and accurately maintained and provided further that the hand mixing is done in a manner meeting with the approval of the Commissioner of Buildings. When a batch mixer is used, the mortar or grout shall be mixed for a period of at least three minutes after all materials for a batch are in the drum. The drum must be completely emptied before the succeeding batch of materials is placed therein.

(b) All grout shall be stirred or worked at frequent intervals before placing to prevent separation of the materials.

### 13-603. PREHYDRATION OF MORTAR AND GROUT

After the original mixing, all mortar and grout shall be emptied into a watertight container and allowed to stand for a period equal to that required for the initial set of the particular brand of Portland cement used. The mortar is again mixed, either by hand or in a mixer, to renew its plasticity before it is used in the masonry.

### 13-604. MORTAR BEDS AND OTHER JOINTS

All brick shall be laid on a full flat bed of mortar. Splitting or furrowing of mortar beds will not be permitted.

### 13-605. REINFORCEMENT AND MINIMUM JOINT THICKNESS

Before the mortar is placed over any bars, such bars shall be in correct position and shall be held without movement until the next course of bricks is laid or until the grouting operation has been completed. The minimum thickness of mortar joints, as related to bar sizes, shall be as follows:

| Bar Size | Minimum Mortar Joint Thickness |
|---|---|
| $\frac{3}{8}''$ or less | $\frac{1}{2}''$ |
| $\frac{1}{2}''$ | $\frac{5}{8}''$ |
| $\frac{5}{8}''$ | $\frac{7}{8}''$ |
| $\frac{3}{4}''$ | $1''$ |
| $\frac{7}{8}''$ | $1\frac{1}{8}''$ |

### 13-606. CONDITION OF EQUIPMENT

All equipment for mixing and transporting the mortar and bricks shall be clean and free from set mortar, dirt or other foreign matter.

### 13-607. REMOVAL OF WATER, LOOSE MATERIAL, DEBRIS, Etc.

All excavations for footings, walls, etc., shall be drained free from water and shall be cleaned in such a manner that all loose sand, dirt and debris will be entirely removed. Before starting footings of reinforced brick masonry, a layer of not less than one inch (1″) of Portland cement mortar shall be spread over the surface of the soil. Immediately thereafter the first course of bricks shall be laid.

### 13-608. JOINING OF WORK

Where fresh masonry joins masonry that is partially set or totally set, the exposed surface of the set masonry shall be cleaned with a wire brush and lightly wetted so as to obtain the best possible bond with the new work. All loose brick and mortar shall be removed.

### 13-609. MONOLITHIC CONSTRUCTION

Where a slab is designed to act monolithically with the supporting girders or beams, the supporting members shall be built without interruption and the work shall be carried on continuously until the slab has been completed.

### 13-610. DISTURBANCE OF COMPLETED WORK

When a girder, beam or slab, or any portion thereof has been completed, such work shall remain undisturbed until the masonry has thoroughly hardened except in the case where work is left off at the end of a day and is recommenced on the following morning or as soon thereafter as possible.

### 13-611. FINISHING OF WORK

All brickwork surfaces shall be finished in a workmanlike manner with a thickness of joint and manner of tooling indicated on the drawings or in the specifications. All brickwork must be brought to proper grade as called for on the drawings.

### 13-612. CLEANING AND TUCKPOINTING

All exterior brickwork shall be thoroughly cleaned and pointed. A five per cent (5%) solution of muriatic acid may be used for cleaning if preceded and followed immediately by copious baths of clean water.

## SECTION 7 — COLD WEATHER REQUIREMENTS

### 13-701. PROTECTION OF BRICKS

All bricks, delivered for use in freezing weather, shall be fully protected by a weathertight covering, such as tarpaulin, Sisalcraft paper, or other weatherproof materials to prevent the accumulation of ice on the bricks. Loose board covering will not be permitted.

### 13-702. HEATING OF SAND

All sand shall be heated in such a manner as to remove all frost, ice and excess moisture. The method and equipment used for heating sand shall be of such character and shall be so handled as will prevent the burning or scorching of the sand.

### 13-703. HEATING OF WATER

All water shall be heated to a temperature of approximately 180 degrees.

### 13-704. HEATING OF BRICKS

When necessary to remove frost or excess moisture, the bricks shall be heated to a temperature of approximately 180 degrees.

### 13-705. PROTECTION OF MORTAR AGAINST FREEZING

After the mortar has been mixed, it shall be maintained at a temperature of not less than 70° F. nor more than 100° F. from the time it is mixed until it has been placed in the masonry. If necessary to obtain this result, the contractor shall use metal mortar boards equipped with a Banjo type gas or oil torch, Sterno flame or with such other heating device as may be necessary to keep the mortar board and the mortar at proper temperatures at all times during the progress of the work.

### 13-706. ENCLOSURES AND ARTIFICIAL HEAT

All finished work shall be protected against freezing for a period of 48 hours by means of enclosures, artificial heat and/or by such other protective methods as will meet with the approval of the Commissioner of Buildings. The use of manure, as a protective covering applied direct to the finished work, will not be permitted.

### 13-707. METHODS AND MATERIALS FOR PROTECTION AGAINST FREEZING

All methods and materials used for the protection of the fresh masonry work against freezing shall be subject to the approval of the Commissioner of Buildings. In general the methods and materials now commonly accepted as suitable for the protection of reinforced concrete construction in freezing weather shall be used. The use of salt or other chemicals intended to lower the freezing point of mortar will not be permitted.

## SECTION 8 — DESIGN — GENERAL CONSIDERATIONS

### 13-801. ASSUMPTIONS

(a) The design of reinforced brick masonry shall be made with reference to working stresses and safe loads, and using the accepted theory of flexure as applied to reinforced concrete.

(b) It is assumed that the steel takes all the tensile stress.

(c) The span length of freely supported beams and slabs shall be the clear span

plus the depth of beam or slab, but shall not exceed the distance between centers of the supports.

(d) Where slabs are built integrally with their supports, the span length may be taken equal to the clear span between faces of supports.

(e) The clear distance between lateral supports of a beam shall not exceed 32 times the least width of compression flange.

(f) The depth of the beam or slab shall be taken as the distance from the centroid of the tensile reinforcement to the compression face.

(g) Center to center distances may be used in the moment determination of all continuous members. Moments prevailing at the faces of support may be used to proportion the members at these sections.

### 13-802. DESIGN LOADS

The provisions for design herein specified are based on the assumption that all structures shall be designed for all dead and live loads coming upon them in accordance with the general requirements of the building code of which this forms a part, with such reductions for girders and lower story columns as are permitted therein.

### 13-803. RESISTANCE TO WIND FORCES

(a) The resisting elements in structures required to resist wind forces shall be limited to the integral structural parts.

(b) The moments, shears, and direct stresses resulting from wind forces determined in accordance with recognized methods shall be added to the maximum stresses which obtain at any section for dead and live loads.

(c) In proportioning the component parts of the structure for the maximum combined stresses, including wind stresses, the unit stresses shall not exceed the allowable stresses for combined live and dead loads provided in Sections 13-304 and 13-305 by more than one-third. The structural members and their connections shall be so proportioned as to provide suitable rigidity of structure.

### 13-804. GENERAL CONSIDERATIONS

(a) The general principles and methods of reinforced concrete design and analysis, including the computations for bond, anchorage, shear and diagonal tension, that are generally recognized and accepted, can be applied also to reinforced brick masonry.

(b) The design formulas, requirements and limitations concerning various types of reinforced concrete structural members shall apply with equal force to all reinforced brick masonry, except those structural types that are specifically mentioned in the following chapters.

## SECTION 9 — REINFORCED BRICK MASONRY COLUMNS

### 13-901. MINIMUM DIMENSIONS OF SHORT COLUMNS

(a) Principal reinforced brick masonry columns in buildings for which the unsupported length is not greater than ten times the least lateral dimension, shall have a minimum thickness of 12 inches and a minimum gross area of 144 sq. in. Posts that are not continuous from story to story shall have a minimum thickness of 8 inches.

(b) For columns built as part of walls or piers, the value for $A_g$ of the column section shall be taken either as a circle, square or rectangle of which the sides are $1\frac{1}{2}$ in. outside the lateral reinforcement. It shall be permissible to design a circular column and to build it as a square column of the same least lateral dimension. In such case, the permissible load on the gross area and the required percentage of reinforcement must be taken as those of the circular column.

## 13-902. UNSUPPORTED LENGTH OF COLUMNS

(a) The unsupported length of R-B-M columns shall be taken as the clear distance between the floor surface and the under side of the deeper beam framing into the column in each direction at the next higher floor level.

(b) For rectangular columns, that length shall be considered which produces the greatest ratio of length to depth of section.

## 13-903. PERMISSIBLE LOAD ON SHORT COLUMNS

The maximum axial load on columns, whose unsupported length is not more than ten times the least lateral dimension, shall be computed by the following formula:

$$P/A_g = 0.20\, f'_b\, (1\text{-}p_g) + 0.6\, p_g f_s + 0.1\, p' d'_s\, A_c/A_g$$

where, $P$ = permissible axial load on a column,
$A_c$ = area of core of column measured to outside of lateral ties,
$A_g$ = gross area of column,
$f'_b$ = ultimate compressive strength of brick masonry,
$f_s$ = nominal working stress in vertical reinforcement (usually taken as 40% of yield point),
$f'_s$ = useful limit stress of lateral reinforcement,
$P_g$ = ratio of area of vertical reinforcement to gross area, $A_g$, of column,
$p'$ = ratio of volume of lateral reinforcement to volume of brick masonry core.

## 13-904. VERTICAL REINFORCEMENT

(a) The ratio ($p_g$) of vertical reinforcement shall be not less than 0.50 per cent nor more than six per cent. The number of bars shall be not less than four, nor the diameter less than ⅜ in.

(b) Where lapped splices are used, the amount of lap shall be sufficient to transfer the working stress by bond, using the values given in Section 304.

(c) Welded splices shall be on butted ends and develop stress in tension that is at least equal to the yield point of the steel used.

(d) The column reinforcement shall be held firmly in its designed position and protected by a covering of masonry not less than 1½ in. thick and not less than that required by the provisions of Section 506.

## 13-905. LATERAL REINFORCEMENT

(a) Lateral ties shall be not less than ¼ in. bars, shaped as a circle, square or rectangle. Splices shall be made by welding or over-lapping the ends at least 1″ and bending toward the center of the column. These bends shall be not less than 1″ in diameter with a return toward the center of the column of not less than 3″.

(b) The maximum center to center spacing of lateral reinforcement shall be 20 diameters, or one-half the diameter or least measurement of the core.

(c) The ratio (p′) of volume of lateral reinforcement to volume of brick masonry core shall be not less than 0.40 per cent.

## 13-906. LONG COLUMNS

(a) The maximum permissible axial load (P′) on reinforced brick masonry columns having a length (h) greater than 10 times but not more than 20 times the least lateral dimension (d) shall be determined by the following formula:

$$P' = P(1.3\text{–}.03h/d)$$

where P is the permissible axial load on a short column, determined according to Section 13-903.

(b) When long columns are eccentrically loaded, the value for P is determined in accordance with Section 908 for a permissible load eccentrically applied on a short column. This value for P is then substituted in the above formula to obtain the maximum permissible load, P′, which may be applied eccentrically on a long column.

### 13-907. BENDING MOMENTS IN COLUMNS

(a) Eccentric loading and conditions of restraint, as in the case of rigid frames or other forms of continuous construction, cause bending moments in columns and should be considered in the design. Particular attention shall be given to unbalanced floor loads and eccentric loading due to other causes.

(b) Wall columns in buildings shall be designed to resist moments produced by (1) loads on all floors, (2) loads on a single exterior bay at one floor level or two adjacent floor levels.

(c) The resistance to bending at any floor level shall be provided by distributing the moment between the columns immediately above and below that floor in proportion to their relative stiffness and conditions of restraint.

### 13-908. STRESSES DUE TO COMBINED AXIAL LOAD AND BENDING

(a) Recognized methods of analysis shall be followed in calculating the stresses due to combined axial load and bending. The design shall be governed by the maximum fibre stress in compression as well as the tensile stress in the vertical bars.

(b) The permissible compressive fibre stress ($f_b$) in eccentrically loaded columns may be computed by the following formula:

$$\frac{f_b = 0.20\, f'_b\,(1-p_g) + 0.6\, p_g f_s + 0.1\, p' f'_s A_c/A_g}{1 + (n-1)p_g} \times \frac{1 + ec/r^2}{1 + .8ec/r^2}$$

where, $e$ = eccentricity in inches of the resultant load.
$c$ = distance in inches from the extreme fibre in compression to the axis through the center of gravity of the section.
$r$ = radius of gyration of equivalent masonry section.
$f_b$, $f'_s$ and $p_g$ are the same as noted in Section 903.

(c) The permissible tensile stress in the vertical reinforcement may equal that normally allowed in flexural members, provided that all splices in tensile steel, at or near the section of maximum moment, are capable of developing the yield point strength.

## SECTION 10 — REINFORCED BRICK MASONRY WALLS

### 13-1001. GENERAL PROVISIONS

(a) Walls shall be designed to resist any lateral or other pressure to which they are subjected, including eccentric loads.

(b) Exterior basement walls, foundation walls, fire walls and party walls shall be not less than eight inches thick.

(c) R-B-M bearing walls shall have a minimum thickness of 1/24 the unsupported height or width, whichever is shorter. Buttresses, built-in columns, or piers may be designed to carry all the vertical loads.

(d) Panel and enclosure walls of R-B-M shall have a thickness of not less than 5 inches and not less than 1/30 the distance between the supporting or enclosing members.

(e) The uppermost 15 ft. of the height of an R-B-M bearing wall shall be not less than 6½ inches in thickness, and for every two stories downward (not to exceed 25 ft.), the minimum thickness shall be increased 1½ inches.

### 13-1002. REINFORCEMENT AND ANCHORAGE

(a) Each layer of reinforcement in R-B-M walls shall be anchored to reinforced brick masonry or concrete floors, columns, pilasters, buttresses and intersecting walls with reinforcement equivalent to that in the wall.

(b) Walls shall be reinforced with an area of steel in both vertical and horizontal directions at least equal to 0.18% of the cross-sectional area of the wall for electrically-welded wire fabric or 0.25% of the area for reinforcement bars. Walls more than 8 inches thick shall have the reinforcement for each direction placed in two layers or planes parallel to the wall face. One layer, placed near the exterior surface, shall consist of ½ to ⅔ of the total required reinforcement and be placed not less than 2 inches nor more than ⅓ the wall thickness from that surface. The remainder of the reinforcement shall be placed not less than one inch nor more than ⅓ the wall thickness from the interior surface.

(c) The allowable working stresses ($f_b$) in R-B-M bearing walls with minimum reinforcement shall be $0.20f'_b$ for walls having a relation of height to thickness of 10 or less. The working stresses for walls having a relation of height to thickness of more than 10, but not exceeding 25, shall be reduced on the basis used for long columns in Section 13-906.

(d) The permissible stresses in R-B-M walls that are eccentrically loaded or subject to bending moments shall be computed by the same method used for columns, described in Section 13-908.

### 13-1003. PIERS AND NARROW WALL SECTIONS

When the unsupported height of piers in R-B-M walls exceeds 10 times their least dimension, they shall be designed as R-B-M columns.

Bearing walls, in which the openings are such as to leave relatively narrow sections with heights exceeding 10 times their least dimension, shall be considered as columns.

### 13-1004. CHASES, RECESSES AND OPENINGS

(a) Chases and recesses shall be permitted as in solid masonry walls.

(b) Openings for doors and windows shall have R-B-M, protected steel or other approved lintels.

## SECTION 11 — FOOTINGS

### 13-1101. LOADS

(a) Reinforced brick masonry footings bearing on soil shall be proportioned as to area on the basis of the total load used in designing the column or wall plus the weight of the column or wall and footing. For computations of moments and shears, an upward reaction per unit area shall be based on the total load (not including the weight of the footing itself) divided by the area.

### 13-1102. STEPPED FOOTINGS

(a) R-B-M footings in which the thickness has been determined by the requirements for shear may be stepped, provided that the shear on any section does not exceed the allowable value, and provided further that the thickness of the footing above the reinforcement at the edge shall not be less than 8 in.

## 13-1103. BENDING MOMENT IN FOOTINGS

(a) The critical section for bending moment in an R-B-M footing which supports an R-B-M column, pedestal or wall, shall be considered to be at the face of the column, pedestal, or wall. For R-B-M footings under plain masonry walls, the critical section shall be assumed as halfway between the middle and the edge of the wall. For footings under metallic bases, the critical section shall be assumed as halfway between the face of the column or pedestal and the edge of the metallic base.

(b) The bending moment at the critical section in a rectangular footing having its length not greater than one and one-half times its width, shall be computed from the load on trapezoids bounded by the line of the critical section for moment, the corresponding parallel edge of the footing, and the two diagonals.

(c) The load on the rectangular portion of the trapezoid shall be considered as applied at its center of gravity. The load on the two triangles of the trapezoid shall be considered as applied at a distance equal to six-tenths of the projection of the footing from the line of critical section for moment.

## 13-1104. SHEARING AND BOND STRESSES

(a) The critical section for shear as a measure of diagonal tension in footings shall be assumed as a vertical section at a distance, d, from the face of the column, wall or pedestal supported by the footing.

(b) The critical sections for bond shall be assumed at the face of the column, wall, or pedestal; at vertical planes where changes occur, and near the end of the bending moment reinforcement.

## 13-1105. TRANSFER OF STRESS AT BASE OF COLUMN

(a) The compressive stress in longitudinal reinforcement at the base of a column shall be transferred to the pedestal or footing by dowels. There shall be at least one dowel for each column bar, and the total sectional area of the dowels shall not be less than the sectional area of the longitudinal reinforcement in the columns. The dowels shall extend into both the column and the pedestal or footing the distance required for lap of longitudinal column bars.

(b) The permissible compressive unit stress on top of the pedestal or footing directly under the column shall not be greater than that determined by formula

$$P_b = 0.25\, f'_b\, (A/A')^{1/3}$$

where, $P_b$ = permissible unit working stress in brickwork on top of a footing under a column.

$A$ = total area of top of footing at base of column.

$A'$ = loaded area of footing at the column base.

(c) In stepped footings, A may be taken as the area of the top horizontal surface of the footing.

## 13-1106. PEDESTALS AND FOOTINGS (PLAIN BRICK MASONRY)

(a) The allowable compressive unit stress on the gross area of a concentrically loaded pedestal shall not exceed $0.25f'_b$. Where this stress is exceeded, reinforcement shall be provided and the member designed as a reinforced brick masonry column.

(b) The depth and width of a pedestal or footing of plain masonry shall be such that the tension in the masonry shall not exceed $.025f'_b$, and the average shearing stress shall not exceed $.02f'_b$ taken on critical sections as determined for reinforced brick masonry footings.

# CHAPTER 14

# BRICK SEWERS

## 1401. BRICK SEWERS

The following discussion of Brick Sewers has been divided into two parts. Part I deals with the importance of sewerage, the history of its development and important factors to be considered in selecting the material of which the sewer will be constructed. Part II is on the design of Brick Sewers and contains examples illustrating the application of design principles.

Vitrified clay or shale sewer pipe has been largely used in this country since the first sewerage systems were installed for the construction of sewers of the smaller diameters (6 to 24 inches) and is considered by most sanitary engineers as the best material available for the purpose. Its record of service during the past seventy-five years amply justifies this rank. Vitrified pipe sewers in sizes up to 24 or 30 inches may be economically constructed in most communities, and for this reason most brick sewers which have been constructed are of diameters of 36 inches or over and very few brick sewers have been constructed which have diameters less than 30 inches.

When reference is made to sewers in the succeeding pages, the reader should bear in mind that diameters of 36 inches and over are implied.

# PART I — ECONOMIES OF SEWERAGE

## 1402. IMPORTANCE OF SEWERAGE

The importance of an adequate and permanent sewerage system to a community is rarely recognized by the majority of its inhabitants and until about the middle of the nineteenth century (1830-1840) no effective measures had been taken to enforce sanitation or to provide adequate sewerage systems in the principal metropolitan centers of the world. It was not until a series of devastating plagues and epidemics of disease visited these centers of population, claiming thousands of lives, that the public was aroused to the point of providing effective sewerage systems which would eliminate the menace to health of unsanitary conditions.

In 1849 cholera broke out in London and before it was stamped out 14,600 deaths were recorded. In 1854 it appeared again claiming 10,675 lives, after which Parliament passed an Act in 1855 under which an adequate sewerage system was installed.

The development of a sewerage system in Paris, as in London, followed an epidemic of cholera which broke out in 1832. The following year surveys were made and a complete sewerage system was planned.

Due to the relative sparsity of population in the cities of this country, sanitary conditions were not as bad as they were in Europe when cholera was sweeping London and Paris and, profiting by the experience in Europe, most of these cities took steps during the latter part of the nineteenth century to provide adequate sewerage systems, although in 1873 an epidemic of yellow fever swept Memphis, claiming 2,000 lives and in 1878, 5,150 deaths occurred from the same cause, following which steps were immediately taken to provide adequate sewerage.

Experience, both in this country and in Europe, over the last seventy-five years has thoroughly vindicated the claims of early engineers and health officers, that the health, and in fact the very existence of metropolitan areas, depend upon effective and adequate

sewerage systems. Messrs. Metcalf and Eddy, the authors of one of the outstanding works on sanitary engineering, may have had this fact in mind when they stated: "The sanitary engineer who neglects to work for the best interests of the public health falls short of the full discharge of his professional duties . . . ".

## 1403. HISTORY OF SEWERAGE DEVELOPMENT

Most histories of the development of sanitary engineering deal primarily with the evolution of the hydraulic design of sewers. When the first sewerage systems were constructed, the principles of hydraulic design as accepted and used today were unknown and various theories were developed and used by the engineers of those days in an effort to estimate the proper sizes and grades for the sewers which were designed to serve a given area.

Many of these theories were later proved to be faulty and unreliable. However, during the last 75 years, the methods of hydraulic design have been reasonably well standardized through research and the study of existing sewerage systems.

In the construction of the first sewers, however, the early engineers turned to a material which was already old in point of service and whose structural qualities had been thoroughly tested. This material was brick which was used in the construction of most of the first sewerage systems and which has amply justified the reliance placed upon it by early engineers. Many old sewers, constructed 50 or 100 years ago, are still in use today.

In 1876 a committee composed of E. S. Chesbrough, Moses Lane, and Dr. C. F. Folsom, which had been commissioned by the authorities of Boston, Massachusetts to make a study and report of the sewerage systems then in existence, reported that Hamburg, Germany "was the first city which had a complete systematic sewerage system throughout according to modern ideas."

The sewerage system of Hamburg was designed by W. Lindley, one of the leading English engineers of his day, and was constructed in 1843. Metcalf and Eddy, in their "American Sewerage Practice", state regarding his design: "It is astonishing to reflect that from the day of Frontinus (the first century A.D.) to that of W. Lindley, there was no marked progress in sewerage."

The sewers in Hamburg are constructed of brick and, according to a report presented by Mr. Chesbrough to the Sewerage Commission of Chicago, "presented a better appearance with regard to both material and workmanship than anything of the kind I have ever had an opportunity to examine."

The first complete sewerage system to be constructed in this country was designed by E. S. Chesbrough who made an investigation and study of the sewers in the principal European cities. The following are extracts from the report of his investigations:

> "London — Brick sewers of over 808 miles; the sewers here have furnished nearly all the materials from which have been elaborated the different theories which now prevail with regard to the best mode of draining towns and cities throughout the world.
>
> "Berlin — This city in many respects more like Chicago than any other I had visited. The foundations of stone for the sewers made the construction very expensive, the bottom and top arches are generally of brick.
>
> "Leicester, England — The sections of the sewers are circular, this form being preferred on account of superior strength, economy, and efficiency. The material used in all sewers is brick, radiated for the smaller ones.
>
> "Carlisle, England — All sewers built of brick."

The Chicago sewers, constructed under Mr. Chesbrough's supervision, were brick and in 1911 Mr. C. D. Hill, Chief Engineer of the Board of Local Improvements of Chicago, stated in an article written by him for the Journal of the Western Society of Engineers: "At this point it may be proper to state although it has been necessary to replace a few of the sewers laid by the village authorities before annexation, because of improper construction, there have been no sewers laid by the City of Chicago, that have been replaced for that reason, and there have been very few instances where it has been necessary to make repairs on account of poor workmanship or materials. Nor have we been able in fifty years to improve on the high standard of workmanship established in the beginning by Mr. Chesbrough."

Subsequent to the construction of the Chicago sewerage system, complete systems were designed and built in all of the principal cities of this country and in almost every instance the engineer in charge chose brick as the material from which to build the sewers. The fact that the early engineers selected brick for their sewers may not be so significant as it would be today in view of the fact that the number of materials available for this purpose was not as great during the period 1850 to 1880 as it is at the present time. However, the fact that many of these sewers are still in use and have functioned practically without maintenance for from 50 to 80 years is remarkable testimony to the quality of the material chosen.

## 1404. REQUIREMENTS OF SEWERS

There are certain basic requirements applicable to all sewers relative to which any proposed installation might well be considered.

These, in the order of their importance are:

(a) Design
(b) Permanence
(c) Cost

### (a) Design

The design of a sewer may be broadly divided into two parts — determination of the size required to carry the sewage which will be delivered to the sewer from the area being drained, referred to in this discussion as hydraulic design; and the determination of the thickness of the sewer walls required to carry the loads (usually earth) which are imposed upon the sewer, referred to as structural design.

It is obvious that a sewer which fails in either of the design requirements listed above cannot properly perform the functions for which it is constructed. For this reason design is placed as the most important consideration in connection with the installation of a sewerage system.

Both the hydraulic and structural design of a sewer depend upon conditions which are not readily determinable in advance of construction. The size of sewer required depends upon the material of which the sewer is constructed, the topography of the land, and the quantity of sewage which a given area may be expected to deliver to the sewer. This latter factor must be determined largely by the judgment and experience of the engineer, based upon all information available as to the probable development of the area which is being sewered. Unforeseen changes in the uses of land, frequently make sewers, although properly designed when constructed, totally inadequate in later years.

It is impossible, of course, to predict accurately the requirements which will be made of a sewer thirty or fifty years after its construction. However, in view of the tremendous expense incident to the reconstruction of sewers in congested areas, no effort should be spared on the part of those in authority to obtain the best design possible. In order to accomplish this, competent sanitary engineers should be employed and a complete sewerage system should be laid out. Sewers installed more or less at random

and without competent engineering advice have always proved very costly to those municipalities which adopted this method of procedure.

The structural design of a sewer is based upon the loads which will be transmitted to the sewer due to the earth fill above it and any super-imposed loads. The exact magnitude of these loads cannot be determined and here again the judgment and experience of the engineer must be relied upon to obtain an economical and safe design.

**(b) Permanence**

Given a proper design, the next consideration in the construction of a sewerage system should certainly be its permanence.

Recently, the argument has been advanced that some of our buildings and structures have been built too well, that they become obsolete before they are worn out and that economical design involves determining the life expectancy of a structure and then building only for that period with the expectation that at the end of that time the structure will have outlived its usefulness and will be replaced.

There is some question as to the soundness of this argument even when applied to structures above ground. However, for sub-surface structures such as sewers, it most certainly does not apply. As previously stated, the health of our modern communities is dependent upon the adequacy and effectiveness of their sewerage systems. The inspection of sewers is difficult, and frequently sewers function for years without maintenance or inspection. When a sewer fails, it may be years before the failure is discovered and during this time, it constitutes a serious menace to the health of the community. The reconstruction of a sewer is a very expensive operation and may require the expenditure of many times the first cost of installation.

The streets of our modern cities are usually paved with hard surfaces and often carry electric railway tracks. The sub-surface structures consist of water mains, gas mains, telephone and electric lighting conduits, sewers, and in some instances heating mains and auxiliary high-pressure water mains.

The failure of a sewer frequently causes the street to cave which besides destroying the pavement may cause the failure of one or more of the other sub-surface structures and serious damage to the adjacent property. The cost of a sewer failure, therefore, cannot be measured simply by the cost of replacing the sewer, but must include also the damage to other property. This fact was realized by the engineers who designed and constructed the first sewerage systems in this country, most of which are still in use today, and through their foresight in building permanent structures, they have saved the municipalities for which they labored many thousands of dollars.

**(c) Cost**

Experience in this country during the past fifty years justifies the conclusion that the permanence of sewers should be given prior consideration to their first cost. However, since most sewers are paid for by money derived from taxes, it is incumbent upon the engineer and the public officials in charge to obtain an adequate and permanent sewerage system at as low a cost as possible.

This does not necessarily mean the adoption of a material, a type of construction, or a size of sewer which has the lowest first cost. As previously stated, correct design and permanence should undoubtedly take precedence over first cost. Also, in determining the cost of a sewerage system, maintenance should be considered. It is palpably poor economy to install sewers at a low first cost if the maintenance of these sewers will be excessive.

Occasionally, in the desire to furnish much needed sewerage, public officials have adopted questionable designs and unreliable materials in the construction of their sewerage systems with a view to providing the greatest number of sewers possible with a limited amount of money. Such practices have inevitably proved disastrous and costly.

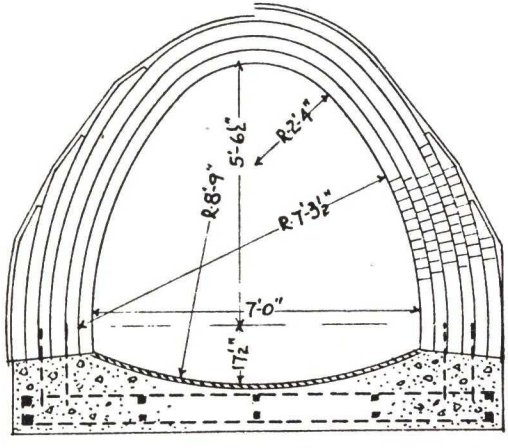
On the other hand, in communities where the sewerage system has been designed by competent engineers and constructed of tried materials, the sewers are functioning today and the costs of replacements, damages, and maintenance have been negligible.

## 1405. DISINTEGRATION OF SEWERS

Sewers are constantly subjected to the attacks of acid or alkaline solutions both in the ground water surrounding the sewers and in the sewage which flows through them. Regarding this action, Metcalf and Eddy state in their "American Sewerage Practice", Volume I, page 451: "Reference has already been made in Chapter XI, in the discussion on concrete pipe, to the possibility of disintegrating action on concrete of seepage from acid or alkali soils. Where such destructive action is likely, consideration must be given to protective methods if concrete or cement mortar are to be used successfully.

"Hydrogen sulphide given off by decomposing organic matter in sewage may form sulphuric acid above the flow line, which will attack some of the calcium compounds in cement, forming soluble calcium sulphate. In forming, this compound expands and scales off the concrete and then gradually dissolves, leaving the aggregate uncemented. Thus the sewer may be seriously damaged. An interesting example of this is the Western Ave. sewer, Chicago, which was damaged by stockyards' sewage giving off hydrogen sulphide. For a new sewer carrying similar sewage, the Sanitary District of Chicago adopted the brick-and-concrete design shown in Fig. 142, in preference to concrete, and at a somewhat higher cost, partly because of the protection from disintegration afforded by the brick arch and tile-lined invert."

The following is a sketch of the Chicago stockyards intercepter referred to above as Figure 142:




STOCKYARDS INTERCEPTER - CHICAGO

*Fig. 33*

In response to inquiries made of the principal cities of the United States, only two or three cities have reported damages to their sewers due to hydrogen sulphide gas and in most instances the damage reported was relatively slight.

An investigation by the authors of the facts in connection with the partial failure of a sewer in Cleveland, conducting sewage having a high acidity from an industrial rayon plant, disclosed that the sections of the sewer which were constructed of brick had not failed, but that other sections constructed of other materials had failed. In

discussing this situation with the Chief Sewer Inspector of the City of Cleveland, who has been connected with the department for approximately forty years, he stated that in his inspections of brick sewers, he frequently found mortar joints in the arches of old sewers which indicated that they had been subject to attack by hydrogen sulphide gas. These joints, for a depth of from one-fourth to one-half inch, contained a plastic putty-like substance which could easily be scraped away with a penknife.

As stated above, when calcium sulphate is formed by the action of sulphuric acid upon concrete or cement mortar, it expands. This expansion together with the wedge shape of the joints in the sewer prevents the decomposed mortar from dropping out of the joint and by its retention in the joint, it acts as an insulator and protects the mortar in the back of the joint from the sewer gas.

The Chief Sewer Inspector of the City of Pittsburgh, has found the same situation existing in some of the older brick sewers of Pittsburgh and in this city no serious difficulties have been experienced due to the effects of hydrogen sulphide gas on brick sewers.

The only sewerage system in this country which has come to our attention in which the effect of hydrogen sulphide gas on brick sewers has necessitated partial reconstruction of the sewers is in Los Angeles.

Regarding this situation, Mr. H. G. Smith, Engineer of Sewer Design, Bureau of Engineering, Los Angeles, California, writes in the March 28th, 1935 issue of "Engineering News Record":

"Severe gas conditions are to be expected in the lower portions of a sewer system totaling 2,900 miles in length with direct flows ranging up to 45 miles long, which are the approximate figures on the Los Angeles system. For many years there has been ample evidence of the serious nature of attack on sewer linings of this city by acid combinations of hydrogen sulphide gas. As a result, particularly in the last six or eight years, experiments constantly have been under way to determine how materials used in the construction of trunk sewers can be protected from these attacks. No discovery has been made of ideal materials nor of methods that can be offered with unqualified recommendations, although the use of fused sulphur silicate seems to hold great promise. The best results thus far in resisting the most intensive gas conditions in the trunk sewers have been obtained by using the clay liners mentioned in the foregoing or hard-burned wire-cut brick able to show less than 10 per cent absorption. Where these liners or bricks of the qualities mentioned have been placed with poured sulphur-silicate joints, no indications of deterioration whatsoever have been observed in a period of more than three years. Individual test specimens placed in the sewer prior to these three-year-old installations still retain their original integrity."

The resistance of brick to the attacks of acids or alkali is well-known and while, as stated in Mr. Smith's article, fused sulphur-silicate joints may not be unqualifiedly recommended in view of the present available data concerning them, they do, at present, present the best solution to the problem of combating the effects of sewer gas in sewers where gas conditions may be expected to constitute a serious menace.

As stated above, such sewers are rare and it is believed that in the great majority of cases ordinary brick sewer construction, using care to obtain thin and well-filled joints on the intrados of the crown, will give satisfactory results and that additional precautions will not be necessary.

## 1406. ABRASIVE ACTION OF SEWAGE ON BRICK SEWERS

It would be difficult to estimate the total miles of brick sewers now in use in this country. In response to a questionnaire sent in 1935 to the sewer departments of the one hundred largest cities in the United States, forty-four cities reported a total of 5924 miles of brick sewers then in use (see Table 46). A large percentage of these sewers are now more than thirty years old.

## TABLE 46

## RESULT OF SURVEY CONDUCTED BY THE BRICK MANUFACTURERS ASSOCIATION IN 1935

| City and State | Total length brick sewers (miles) | City and State | Total length brick sewers (miles) |
|---|---|---|---|
| Akron, Ohio | 49 | Oklahoma City, Okla. | 0.5 |
| Baltimore, Md. | 148 | Philadelphia, Pa. | 1600 |
| Bronx, N. Y. | 11 | Pittsburgh, Pa. | 90.8 |
| Brooklyn, N. Y. | 193 | Portland, Ore. | 18.9 |
| Canton, Ohio | 20 | Providence, R. I. | 105 |
| Chattanooga, Tenn. | 14.2 | Queens, N. Y. | 10 |
| Chicago, Ill. | 2300 | Richmond, Va. | 79.94 |
| Columbus, Ohio | 100 | Rochester, N. Y. | 20.5 |
| Dallas, Texas | 3 | Sacramento, Calif. | 5.6 |
| Dayton, Ohio | 15 | Salt Lake City, Utah | 7 |
| Detroit, Mich. | 350 | San Francisco, Calif. | 30 |
| Fall River, Mass. | 23 | Scranton, Pa. | 15 |
| Flint, Mich. | 2.25 | Seattle, Wash. | 19.68 |
| Fort Wayne, Ind. | 20 | Spokane, Wash. | 0.2 |
| Hartford, Conn. | 50 | Syracuse, N. Y. | 35 |
| Jersey City, N. J. | 76.5 | Tacoma, Wash. | 0.23 |
| Los Angeles, Calif. | 22.11 | Toledo, Ohio | 153.3 |
| Lowell, Mass. | 25.5 | Tulsa, Okla. | 2.627 |
| Minneapolis, Minn. | 106.3 | Wichita, Kans. | 12 |
| Nashville, Tenn. | 52 | Wilkes-Barre, Pa. | 3.5 |
| Newark, N. J. | 69 | Wilmington, Del. | 5.3 |
| New Haven, Conn. | 44.41 | Youngstown, Ohio | 15 |
| | | TOTAL | 5924.34 |

There are few records that indicate the quality of the brick used in the construction of these old sewers, but such as are available show that, as a rule, they were constructed of the brick produced locally.

J. W. McBurney, in reporting strength, water absorption, and weather resistance of building brick produced in the United States, lists averages from thirteen principal cities and twenty-six districts (see Table 47). Of the cities reported, the lowest absorption by five-hour boil are Pittsburgh brick with 7.7%, and the highest are Detroit brick with 19.5%. The average for the thirteen cities is 13.3%.

It is therefore probably safe to conclude that the majority of brick sewers constructed in this country prior to 1900 were built of brick having an absorption (five-hour boil) of more than 10%.

Relatively few people ever see the inside of a sewer after it is constructed. Few engineering structures receive less attention or are inspected less frequently. Consequently, data regarding the abrasive action of sewage and the erosion of sewer inverts are surprisingly scarce, and discussions of this subject are more often based upon theoretical considerations than upon actual performance records.

TABLE 47

WEIGHTED AVERAGES OF STRENGTH AND WATER ABSORPTION FOR HARD BRICK

| City | Av. compressive strength (lb./sq. in.) | Av. Modulus of rupture (lb./sq. in.) | Av. water absorption (%) 5-hr. boil |
|---|---|---|---|
| Boston, Mass | 8240 | 1360 | 10.6 |
| Hudson Valley | 4740 | 745 | 17.7 |
| Baltimore, Md | 5990 | 1330 | 12.3 |
| Philadelphia, Pa | 6580 | 855 | 14.5 |
| Pittsburgh, Pa | 11060 | 2055 | 7.7 |
| Cleveland, Ohio | 9820 | 1185 | 11.1 |
| Detroit, Mich | 4330 | 900 | 19.5 |
| Chicago, Ill | 3450 | 1265 | 19.0 |
| St. Louis, Mo | 11040 | 1465 | 9.1 |
| Denver, Colo | 5050 | 620 | 13.5 |
| Los Angeles, Calif | 3690 | 600 | 14.2 |
| Richmond, Va | 9350 | 1315 | 11.1 |
| Washington, D. C | 5700 | 875 | 12.9 |

**Review of Investigation** — One of the most extensive investigations of the wear on sewer inverts, of which there is a record, was conducted during 1909 and 1910 by Metcalf and Eddy, sanitary engineers of Boston, who inspected the brick sewers of Worcester, Mass. Brick samples were taken from some of the sewers, of which the following are typical:

(a) Brick taken from a 30- by 45-inch egg-shaped section built by contract in 1874. The masonry of this sewer was constructed of two rings of sandstruck brick of 20 to 30% absorption, laid in Rosendale cement mortar. The brick were taken from a section where the grade is 6.94%. The velocity in this section (based on Kutter's formula, $n = 0.015$) at two-thirds full is 22 feet per second.

The upstream ends of the brick were worn more than the downstream ends. In some instances, the downstream ends were worn off 2 to 2½ inches. The mortar was very sandy and comparatively soft and had been washed out for a considerable depth below the surface of the brick.

(b) Another sample was taken from a 48- by 72-inch egg-shaped section, built in 1872 by contract, of 8-inch brickwork laid in Rosendale cement mortar. These brick were taken from the invert on a section where the grade was 2.9%, and the estimated velocity flowing two-thirds full was 20 feet per second. The brick were very hard and dense, probably having an absorption of 8 to 12% and were worn very smooth almost to a polish. The small end in each case was the upstream end. In some instances, 1½ to 2 inches had been worn off at the upstream end of these brick.

Regarding the Worcester sewers, the investigators state:

> "The brick invert was found to be badly worn in all sections where the velocity flowing two-thirds full exceeded 8 or 9 feet per second, estimated by Kutter's formula with $n = 0.015$. In some sections, where the estimated velocity amounted to 12 or 13 feet per second, the first course of brick in the invert was worn through in places, and the second course was partly worn."

The same investigators, however, on a similar investigation conducted at Louisville make the following statement:

> "It is interesting to compare the experience gained at Worcester with information obtained at Louisville from an inspection of old brick sewers. Where the velocity was high, there was but little wear of the brick, while at Worcester sewers having apparently the same velocity showed serious wear. The explanation is that, at Worcester, the street detritus contained a large quantity of quartz sand coming from streets which for many years were surfaced with gravel. There were also large deposits of sand and gravel in the city, and the soil as a whole contained a large amount of quartz. In spite of the large number of catch-basins in use, considerable quantities of sand and gravel entered the sewers, and as the detritus was carried along by the flow of sewage, the invert brick were worn by the harder material. At Louisville, the soil is composed of clay and disintegrated limestone, and the streets were surfaced with crushed limestone which, for the most part is softer than the sewer brick. Even in sewers constructed of relatively soft brick, those testing between 24% and 30% absorption, there appeared to be but little wear from the velocities which at Worcester had caused serious wear. Although doubtless the detritus washed along the inverts at Louisville caused some wear, the attrition was much more effective upon the detritus itself than upon the sewer brick."

A report of the Metropolitan Sewerage Commission of New York contains the following statement regarding the erosion of sewer inverts.

> "Few cases where the brick of the invert were actually worn away were found. In a few places in the upper west side of Manhattan, the upstream edges of the brick were rounded off as a result of high velocity of sewage. In a large number of sewers, the mortar was worn from the joints in the brickwork of the invert. Sometimes the mortar has been worn away only to a slight depth while at other places it has been cut out by the sewage to the full depth of the brick."

E. A. Herman, in 1904, reported on the effects of abrasion on the sewers in St. Louis. The sewers inspected were combined sewers on grades ranging from 0.2 to 2.0%, averaging about 0.5% for sewers more than 5 feet in diameter and about 1% for those of smaller sections. Mr. Herman reports that vitrified clay pipe showed no appreciable wear after about 35 years' use; vitrified brick inverts showed no appreciable wear after about 12 years, the oldest vitrified brick sewers inspected; and inverts of ordinary sewer brick showed some wear after about three years and from two to four inches wear after a use of 30 years.

The absorptions of vitrified brick and ordinary sewer brick, as referred to by Mr. Herman, are not given in his report. Vitrified brick, however, are described by him as "building foundation vitrified brick."

During the early part of 1937, the James H. Herron Company, Cleveland, Ohio, inspected three brick sewers in Cleveland for the Brick Manufacturers Association of America. All of the sewers inspected are combined sewers. A summary of the report of these inspections and of the test conducted on brick samples taken from the old sewers is given.

### TEST NO. 1

A sample consisting of six whole brick was taken from the East 14th Street sewer, approximately 100 feet south of Scovill Avenue. The overlying pavement consists of

brick laid directly on compacted earth. The original pavement was apparently filled with cement grout, but subsequent repairs have been made using bituminous filler. Traffic includes numerous heavily loaded trucks bound for the adjacent market district and trucking terminals.

The sewer at this point is egg-shaped, 27 by 23¼ inches inside dimensions, and is constructed of a single ring of common brick, 4 inches thick. The sewer is 16 feet below grade in sandy soil and has been in service for 55 years. The grade of the section inspected is 0.4%, and the estimated maximum velocity is 4.45 feet per second.

Tests of five whole brick taken from this sewer gave an average absorption (5-hour boil) of 18.6%, an average compressive strength on edge of 3759 pounds per square inch, and an average modulus of rupture of 709 pounds per square inch.

Regarding the condition of the brick, the report states:

> "The inner faces of the brick were not badly worn and, in general, the brick appeared to be in good condition.
>
> "A manhole, constructed at the same time as the sewer, was also inspected; the condition of the brick was noted to a depth of 4 feet below grade. No evidence of disintegration on the part of the brick was noted. When tested by blows of a hammer, the brick appeared to be as hard as those taken from the sewer. Mortar between the brick had worn away through the action of water and frost, in some places to the depth of one inch. That remaining in place could be readily removed with hammer and chisel."

## TEST NO. 2

A sample consisting of five brick was taken from the East 66th Street sewer at the intersection of Hough Avenue. The overlying pavement is sheet asphalt. Traffic is principally fast-moving passenger cars but also includes some medium-weight trucks.

The sewer is an egg-shaped, 27 by 23¼ inches inside dimensions, constructed of common brick, 4 inches thick above and 8 inches thick below the spring line. The sewer is 25 feet below grade and has been in service for 57 years.

The grade of the section inspected is 1.5%, and the estimated maximum velocity is 8.84 feet per second. This sewer connects with a larger sewer built in 1876.

Tests of five brick taken from the sewer gave an average absorption (five-hour boil) of 17.8%, an average compressive strength on edge of 2860 pounds per square inch, and an average modulus of rupture 726 pounds per square inch.

Regarding the condition of the brick, the report states:

> "The sewer south of Hough Avenue was carrying water to a depth of 8 inches when inspected. The brick therein appeared to be in good condition. No large cracks were noticed, and no abrasion was evident on the brick and mortar above the water. The ease, however, with which brick comprising the sample were removed indicated a weak bond between the mortar and brick. Brick taken from the lower part of the sewer from beneath the running water were worn on the inner exposed base, but not sufficiently to weaken the structure. The wear was estimated at not exceeding 0.05 inch average depth, except at the exposed ends, where water and sludge cascaded from the smaller into the larger sewer. Measurements of the wear on these brick follow:
>
> Brick No. 3 worn from 0.30 inch to 0 in 1.0 inch.
>
> Brick No. 4 worn from 0.28 inch to 0 in 2.0-inch length.
>
> Brick No. 5 worn from 0.14 inch to 0 in 0.90 inch.
>
> At this point the mortar between the brick was worn away to a depth of 1.10 in. for a distance of at least 8 inches."

### TEST NO. 3

A sample consisting of five brick was taken from the Addison Road sewer. The overlying pavement is asphalt and the top of the sewer is 16 feet below grade. The sewer is circular in section, 9 feet inside diameter, constructed of three rings of common brick, 12 inches thick, and has been in service for 49 years.

The grade of the section inspected is 0.1% and the estimated velocity flowing half full is 5.42 feet per second.

Tests of five whole brick taken from this sewer gave an average absorption (5-hour boil) of 16.5%, average compressive strength 3667 pounds per square inch, average modulus of rupture 705 pounds per square inch.

Regarding the condition of the brick, the report states:

> "Pitting was noted on a small area on the west side of the sewer about 10 feet south of the manhole. The depth of pitting was estimated not to exceed ⅛ inch. No other evidence of wear was noted on the brick, and no cracks were observed in the brickwork. In a few places in the lower part of the sewer, the mortar was worn out to a depth of ¼ inch. Assuming routine maintenance, such as repointing where necessary, the sewer would be classed as in excellent condition. The brickwork of a manhole constructed at the same date as the sewer was also inspected and was found to be in good condition."

These data are in accordance with the findings of other investigators and they indicate that, under conditions similar to those to which the sewers inspected have been subjected, hard-burned brick of relatively high absorption, when built into a sewer invert, will show no appreciable abrasion after a period of from fifty to sixty years.

## 1407. MAINTENANCE OF BRICK SEWERS

In response to a questionnaire sent to the city engineers of the 100 largest cities in the United States, one engineer writes regarding the maintenance of brick sewers in his city: "Some of the brick sewers that were laid with lime mortar joints prior to about 1890 resulted in partial reconstruction, but apart therefrom maintenance costs of brick sewers are practically negligible."

A study of the history of the development of sewerage systems, both in this country and in Europe, presents overwhelming evidence of the permanence of brick sewers and of their satisfactory performance with negligible maintenance.

## 1408. COST OF BRICK SEWERS

A summary of the opinions of engineers throughout the United States as to the relative cost of brick sewers as compared to sewers constructed of other materials indicates that the impression prevails that the cost of brick sewers is considerably above the cost of comparable sewers constructed of other materials.

This impression is not entirely accurate nor can it be verified by facts. In the first place, as pointed out by Metcalf and Eddy in "American Sewerage Practice": "A comparison of alternate types of sewers should be made on a basis as comparable as possible, particularly if competitive bids are to be taken."

In view of the superior qualities of brick which make it particularly adaptable for sewer construction, especially its resistance to abrasion and attacks by acids, materials which are often considered as alternates for brick are by no means comparable from the standpoint of permanence or future maintenance costs, and these items should undoubtedly be considered when first costs are compared.

A difference frequently exists also in the construction details of alternate designs. Due to the method of constructing a brick sewer in place in the trench, it is essential to

the successful completion of the work that a uniform bearing be provided for the sewer barrel for its entire circumference below the spring line. This method of construction, with good workmanship, guarantees a sewer which will be sound structurally and due to the method of construction, it is impossible to slight this essential detail.

In the construction of sewers of pre-cast pipe, however, this bearing is not always specified and frequently is not obtained in actual practice. It is not an unusual sight to see pipe sewers lowered into a trench excavated full width for its entire depth and back filled only with materials excavated from the trench.

A sewer constructed by this method cannot be considered in any sense comparable to a brick sewer and with the unequal and inadequate bearing provided for the pipe, it cannot be expected to develop its full structural strength and may (as has happened in many instances) fail under the full weight of a saturated back fill.

The structural details (Page 322) have been used by the authors in the construction of sewers in an effort to obtain designs which are comparable from a structural standpoint although with the additional thickness usually provided in brick sewers, the brick sewer illustrated would probably have a greater strength than most pipe sewers.

## PART II — BRICK SEWER DESIGN

### 1409. HYDRAULIC DESIGN

#### (a) Quantity of Sewage

The quantity of sanitary sewage which a given area may be expected to deliver to the sewers has been found to bear a close relationship to the water consumption of the area. There are other factors, however, which must be taken into consideration, such as industrial wastes and infiltration of ground water. Industrial wastes must be determined largely by a study of the industries themselves and will be affected by the availability of raw water or a very low rate for the filtered supply.

In preparing designs for sanitary trunk sewers, it is customary to itemize as follows:

1. Sewage from residential area, gallons per capita per 24 hours.
2. Sewage from commercial area, gallons per acre per 24 hours.
3. Industrial wastes from industrial area, gallons per acre of industrial area per 24 hours.
4. Ground water from district area ultimately to be served, gallons per acre per 24 hours and/or gallons per mile of sewer per 24 hours.

The quantity of storm water which may be expected to reach the sewers from the area tributary to them is a function of the total area drained, the intensity of the rainfall, and the imperviousness and grade of the drainage area.

Various empirical formulae have been developed taking into account the above factors in an effort to estimate the total quantity of storm water which sewers should be designed to carry. The formulae of Hawksley, Adams, McMath, Burkli-Ziegler, Parmley, and Gregory are the better known empirical formulae. At the present time, however, a method of estimating known as the rational method is used by the great majority of sanitary engineers.

In computing the runoff (storm water which reaches the sewers) by this method, the following formula is used:

$$q = ciA$$

In the above formula, q is the runoff in cubic feet per second, c is a coefficient representing the ratio of runoff to rainfall which depends upon the topography of the area and the imperviousness of its surface, i the intensity of rainfall in cubic feet per

second per acre (practically equal to rate of rainfall in inches per hour) and A the measured area in acres tributary to the sewer.

The intensity of rainfall for various intervals of time is usually determined for different localities by means of a chart on which are plotted the actual intensities of the principal storms as recorded by the United States Weather Bureau over a period of years. A curve is then drawn through the points of maximum intensity, usually omitting the most severe storms, occurring only at infrequent intervals, which is known as the time intensity rainfall curve for that area.

If this curve is plotted with intensities as ordinates and time in minutes as abscissas, a runoff curve in cubic feet per second per acre may then be plotted by multiplying the ordinates of the rainfall curve by the proper value of c. Figure 34 is an example of such a curve, which was prepared by the Cuyahoga County Sanitary District (Cleveland, Ohio).

A complete discussion of the rational method of estimating storm water flows is presented in Chapter 7, Volume 1, Second Edition of "American Sewerage Practice" by Metcalf and Eddy.

**(b) Flow in Sewers**

The formula for determining the flow in sewers in common use at the present time is the Chezy formula which was announced by Brahms and Chezy in 1775. This formula is:

$$v = C\sqrt{RS}$$

In this formula, v is the velocity of flow in feet per second, C a coefficient which varies inversely with n, the smoothness factor of the wall of the channel, directly with the hydraulic radius (area of water cross-section divided by length of wetted perimeter) and to a slight degree with the slope. R is the hydraulic radius, and S is the slope of the hydraulic gradient.

A great deal of experimentation has been conducted to determine the proper values of C to be used in the above equation. The formula most widely used for C is known as the Kutter formula and is based upon the experiments of two Swiss engineers, Ganguillet and Kutter, of Berne. The Kutter formula for C is:

$$C = \frac{41.65 + \dfrac{0.00281}{s} + \dfrac{1.811}{n}}{1 + \left(41.65 + \dfrac{0.00281}{s}\right)\sqrt{\dfrac{n}{R}}}$$

n, as used in this formula, is a coefficient of roughness which depends upon the roughness of the channel.

Ganguillet and Kutter recommended a value of n equal to 0.013 for "channels lined with ashlar or neatly joined brickwork." The following values of n are taken from a table recommended by Robert E. Horton and published in "Engineering News-Record", Vol. 75, Pg. 373.

**Values of n to be used with Kutter's Formula**

| Material | Best | Good | Fair | Bad |
|---|---|---|---|---|
| Vitrified sewer pipe | 0.01 | 0.013 | 0.015 | 0.017 |
| Brick Sewers | 0.012 | 0.013 | 0.015 | 0.017 |
| Concrete pipe | 0.012 | 0.013 | 0.015 | 0.016 |

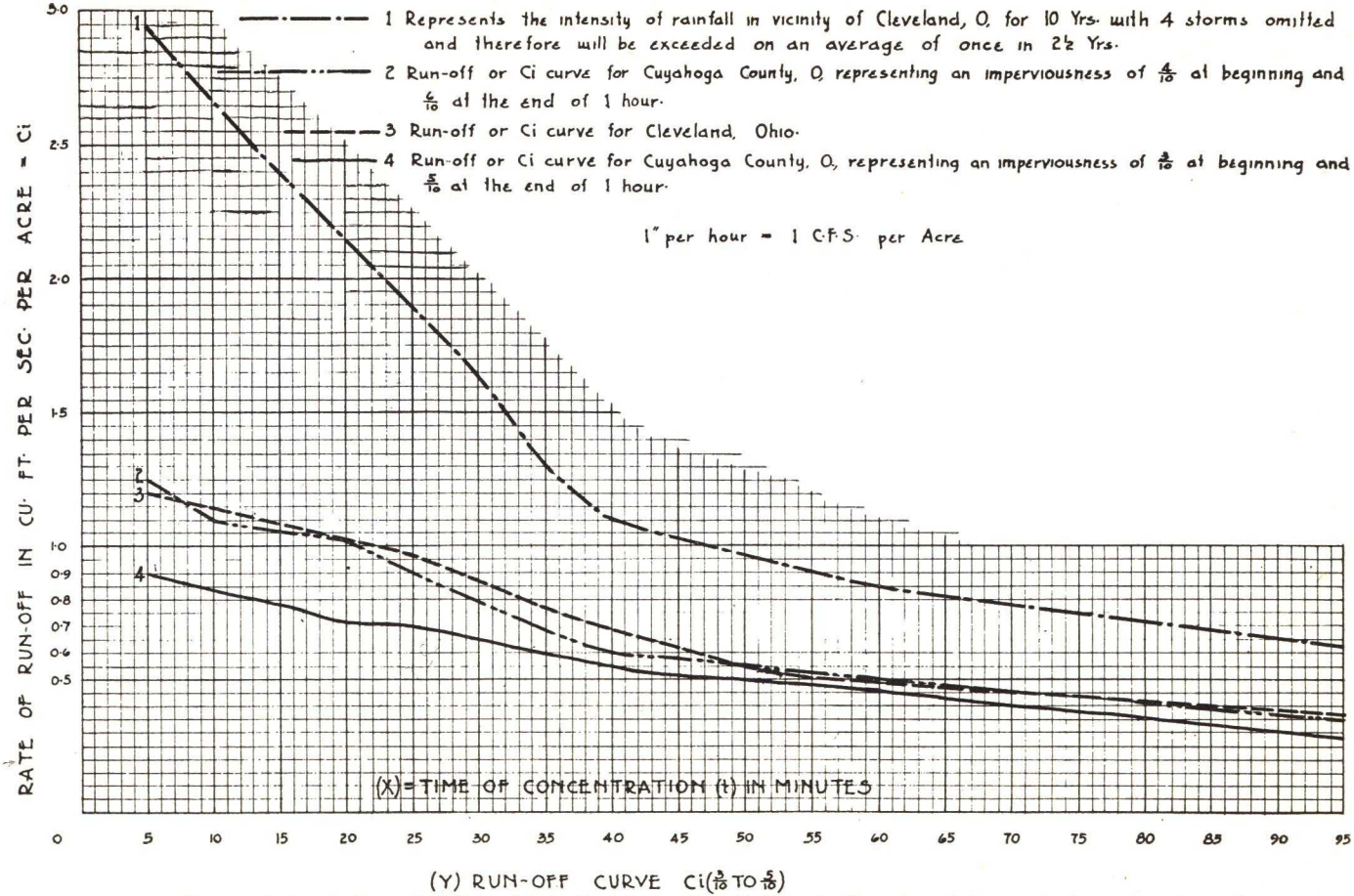



Diagram Showing Rate of Run-off (Ci) in C.F.S. Per Acre for Different Periods of Concentration For Typical Residential Districts

*Figure 34*

The value of n which is used at the present time by most engineers in the design of brick sewers varies from 0.012 to 0.015. In the opinion of the authors, the value of 0.013 may safely be used and if good workmanship is employed in the construction of the sewers and joints in the invert are properly pointed, observations of sewers which have been constructed indicate that this value may easily be obtained.

Table No. 48 taken from Daugherty's "Hydraulics", gives values of C computed from Kutter's formula for slopes ranging from 0.00005 to 0.01, for values of n ranging from 0.009 to 0.035, and for values of R ranging from 0.2 to 15.

Regarding Kutter's formula, Professor Daugherty states: "From the range of experiments upon which it is based, Kutter's formula would appear to be applicable for values of R up to 10 feet, for velocities up to 10 feet per second and for slopes greater than 0.0001. Outside of these limits, reliable data are lacking and Kutter's formula should be used with caution."

Other formulae have been developed and are used to some extent to determine the flow in pipes; these formulae are of the same general form as the Chezy formula. However, since it is recognized that velocity does not vary exactly as the square root of R or S, different exponents are used. The Manning, Bazin, and Hazen and Williams formulae are typical of the exponential formulae.

In order to simplify the computation of discharge by the Chezy and Kutter formulae, various charts have been prepared from which may be obtained for a given cross-section and value of n, the discharge in cubic feet per second for any diameter on any slope. Figure 35 is such a chart prepared by Metcalf and Eddy for a circular sewer having a coefficient of roughness n = 0.013. Similar charts for a large range of cross-sections, slopes, and values of n have been prepared by Mr. John H. Gregory and are used extensively by sanitary engineers.

## 1410. STRUCTURAL DESIGN

The first steps in the structural design of a sewer are the selection of the proper cross-section and materials of which the sewer is to be constructed. Regarding these items, Metcalf and Eddy make the following statement in "American Sewerage Practice", page 62:

"Aside from the hydraulic properties, such considerations as the method of construction, character of foundation, available space, and stability may be instrumental in determining the best type of sewer section to adopt for a given case.

"The selection of the proper thickness of masonry for a given size of sewer, unless determined in the light of experience with similar structures, should be the result of a careful consideration of the forces to be encountered and an analysis of the stresses as determined by the best available methods. This applies particularly to the larger sewers, 6 ft. in diameter and over.

"A study of existing sewers is one of the best guides to safe construction although not necessarily the most economical construction. Empirical formulae founded on experience have some value but should not be depended upon without an adequate analytical check.

"The proper selection of the materials of construction involves not only a comparison of the cost of one material with that of another but also a consideration of the relative life and wearing qualities of the materials. The latter applies especially to the materials used for the lining of the invert.

"In some localities, the erosion of sewer inverts has been a serious problem responsible for the failure of the entire structure. To resist this wear, a lining of vitrified brick has been found satisfactory in some cases."

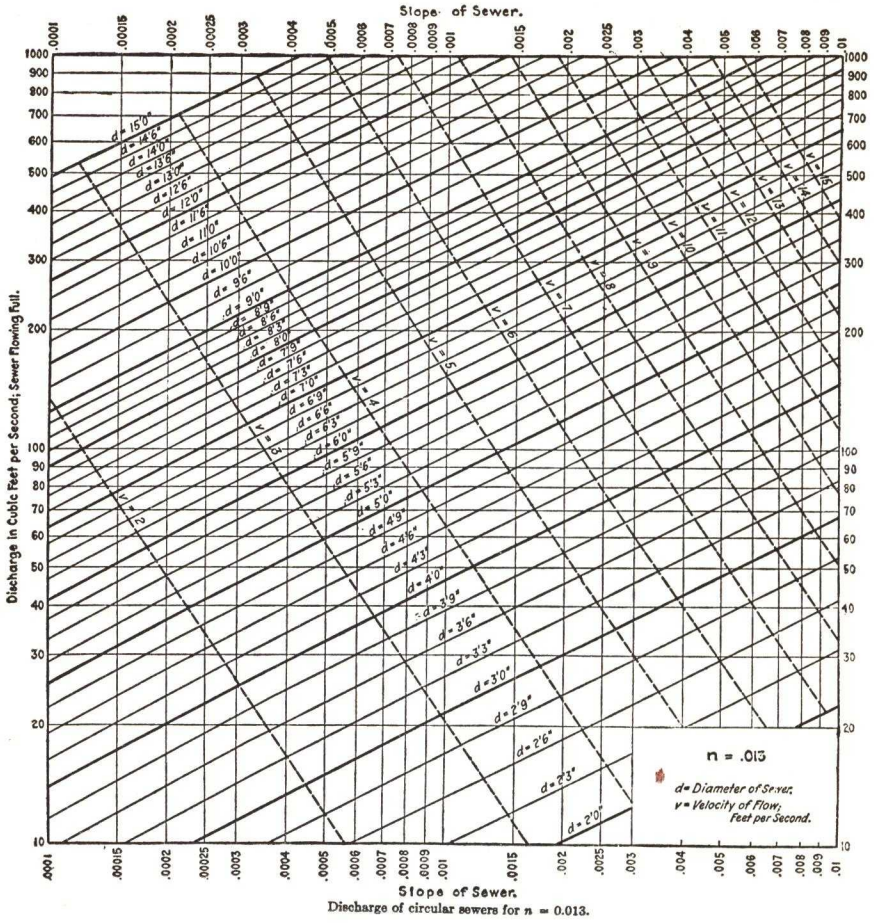
## TABLE NO. 48

## VALUES OF C COMPUTED FROM KUTTER'S FORMULA

| Slope s | n | Hydraulic mean depth, R, in feet | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 0.2 | 0.4 | 0.6 | 0.8 | 1 | 1.5 | 2 | 3 | 4 | 6 | 8 | 10 | 15 |
| 0.00005 | 0.009 | 100 | 124 | 139 | 150 | 158 | 173 | 184 | 198 | 207 | 220 | 228 | 234 | 244 |
| | 0.010 | 87 | 109 | 122 | 133 | 140 | 154 | 164 | 178 | 187 | 199 | 206 | 212 | 220 |
| | 0.011 | 77 | 97 | 109 | 119 | 126 | 139 | 148 | 161 | 170 | 182 | 189 | 195 | 205 |
| | 0.012 | 68 | 88 | 98 | 107 | 114 | 126 | 135 | 148 | 156 | 168 | 175 | 181 | 189 |
| | 0.013 | 62 | 79 | 90 | 98 | 104 | 116 | 124 | 136 | 145 | 156 | 163 | 169 | 179 |
| | 0.015 | 51 | 66 | 76 | 83 | 89 | 99 | 107 | 118 | 126 | 137 | 144 | 149 | 158 |
| | 0.017 | 44 | 57 | 65 | 71 | 77 | 87 | 94 | 104 | 111 | 122 | 129 | 134 | 142 |
| | 0.020 | 35 | 46 | 53 | 59 | 64 | 72 | 79 | 88 | 95 | 105 | 111 | 116 | 125 |
| | 0.025 | 26 | 35 | 41 | 46 | 49 | 57 | 62 | 71 | 77 | 85 | 91 | 96 | 104 |
| | 0.030 | 21 | 28 | 33 | 37 | 40 | 47 | 51 | 59 | 64 | 72 | 78 | 82 | 90 |
| | 0.035 | 18 | 24 | 28 | 31 | 34 | 40 | 44 | 50 | 56 | 63 | 68 | 72 | 79 |
| 0.0001 | 0.009 | 112 | 136 | 149 | 158 | 166 | 178 | 187 | 198 | 206 | 215 | 221 | 226 | 233 |
| | 0.010 | 98 | 119 | 131 | 140 | 147 | 159 | 168 | 178 | 186 | 195 | 201 | 205 | 212 |
| | 0.011 | 86 | 106 | 118 | 126 | 132 | 144 | 151 | 162 | 169 | 178 | 184 | 188 | 195 |
| | 0.012 | 76 | 95 | 105 | 114 | 120 | 130 | 138 | 149 | 155 | 164 | 170 | 174 | 181 |
| | 0.013 | 69 | 86 | 96 | 103 | 109 | 120 | 127 | 137 | 143 | 152 | 158 | 162 | 169 |
| | 0.015 | 57 | 72 | 81 | 88 | 93 | 103 | 109 | 119 | 125 | 134 | 139 | 143 | 150 |
| | 0.017 | 48 | 62 | 70 | 76 | 81 | 89 | 96 | 104 | 111 | 119 | 124 | 128 | 135 |
| | 0.020 | 39 | 50 | 57 | 63 | 67 | 75 | 81 | 89 | 94 | 102 | 107 | 111 | 118 |
| | 0.025 | 29 | 38 | 44 | 48 | 52 | 59 | 64 | 71 | 76 | 84 | 88 | 92 | 98 |
| | 0.030 | 23 | 31 | 35 | 39 | 42 | 48 | 53 | 59 | 64 | 71 | 75 | 78 | 85 |
| | 0.035 | 19 | 25 | 30 | 33 | 35 | 41 | 45 | 51 | 55 | 61 | 66 | 69 | 75 |
| 0.0002 | 0.009 | 121 | 143 | 155 | 164 | 170 | 181 | 188 | 200 | 205 | 213 | 218 | 222 | 228 |
| | 0.010 | 105 | 125 | 138 | 145 | 151 | 162 | 170 | 179 | 185 | 193 | 198 | 201 | 207 |
| | 0.011 | 93 | 112 | 122 | 131 | 136 | 146 | 154 | 163 | 168 | 176 | 182 | 185 | 190 |
| | 0.012 | 83 | 100 | 111 | 118 | 123 | 133 | 140 | 149 | 155 | 162 | 167 | 170 | 176 |
| | 0.013 | 74 | 91 | 100 | 107 | 113 | 122 | 129 | 137 | 143 | 150 | 155 | 158 | 164 |
| | 0.015 | 61 | 76 | 85 | 91 | 96 | 105 | 111 | 119 | 125 | 132 | 137 | 140 | 145 |
| | 0.017 | 52 | 65 | 73 | 79 | 83 | 91 | 97 | 105 | 111 | 117 | 122 | 125 | 131 |
| | 0.020 | 42 | 53 | 60 | 65 | 69 | 77 | 82 | 89 | 94 | 100 | 105 | 108 | 113 |
| | 0.025 | 31 | 40 | 46 | 50 | 54 | 60 | 64 | 72 | 76 | 82 | 87 | 89 | 95 |
| | 0.030 | 25 | 32 | 37 | 41 | 44 | 49 | 54 | 59 | 63 | 69 | 73 | 76 | 82 |
| | 0.035 | 21 | 27 | 31 | 34 | 37 | 42 | 45 | 51 | 55 | 60 | 64 | 67 | 72 |

## TABLE NO. 48 (Continued)

| Slope s | n | Hydraulic mean depth, R, in feet | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 0.2 | 0.4 | 0.6 | 0.8 | 1 | 1.5 | 2 | 3 | 4 | 6 | 8 | 10 | 15 |
| 0.0004 | 0.009 | 126 | 147 | 157 | 166 | 172 | 183 | 190 | 199 | 204 | 211 | 215 | 219 | 224 |
| | 0.010 | 110 | 129 | 140 | 148 | 154 | 164 | 170 | 179 | 184 | 191 | 196 | 199 | 203 |
| | 0.011 | 97 | 115 | 126 | 133 | 138 | 148 | 154 | 162 | 168 | 175 | 179 | 183 | 187 |
| | 0.012 | 87 | 104 | 113 | 121 | 125 | 135 | 141 | 149 | 154 | 161 | 165 | 168 | 172 |
| | 0.013 | 78 | 94 | 103 | 110 | 115 | 124 | 130 | 138 | 142 | 149 | 153 | 157 | 162 |
| | 0.015 | 65 | 79 | 87 | 93 | 98 | 106 | 112 | 119 | 124 | 130 | 135 | 138 | 143 |
| | 0.017 | 54 | 68 | 75 | 81 | 85 | 93 | 98 | 105 | 110 | 116 | 120 | 123 | 128 |
| | 0.020 | 44 | 55 | 62 | 67 | 70 | 78 | 83 | 89 | 94 | 99 | 104 | 107 | 110 |
| | 0.025 | 32 | 42 | 47 | 51 | 55 | 61 | 65 | 71 | 76 | 81 | 85 | 88 | 92 |
| | 0.030 | 25 | 33 | 38 | 42 | 45 | 50 | 54 | 59 | 63 | 69 | 73 | 75 | 80 |
| | 0.035 | 21 | 27 | 31 | 35 | 37 | 42 | 45 | 51 | 55 | 60 | 64 | 66 | 70 |
| 0.0010 | 0.009 | 129 | 150 | 161 | 169 | 175 | 184 | 191 | 199 | 204 | 211 | 214 | 218 | 222 |
| | 0.010 | 113 | 131 | 142 | 150 | 155 | 165 | 171 | 179 | 184 | 190 | 194 | 197 | 202 |
| | 0.011 | 99 | 117 | 127 | 134 | 139 | 149 | 155 | 163 | 168 | 174 | 178 | 181 | 186 |
| | 0.012 | 89 | 105 | 115 | 122 | 127 | 136 | 142 | 149 | 154 | 160 | 164 | 167 | 171 |
| | 0.013 | 81 | 96 | 104 | 111 | 116 | 124 | 134 | 138 | 142 | 149 | 152 | 155 | 160 |
| | 0.015 | 66 | 80 | 88 | 94 | 99 | 108 | 112 | 119 | 124 | 130 | 134 | 136 | 141 |
| | 0.017 | 57 | 69 | 76 | 82 | 86 | 93 | 98 | 105 | 110 | 116 | 120 | 122 | 127 |
| | 0.020 | 45 | 56 | 63 | 68 | 71 | 78 | 83 | 89 | 93 | 99 | 103 | 105 | 110 |
| | 0.025 | 34 | 43 | 48 | 52 | 56 | 62 | 66 | 71 | 75 | 81 | 85 | 87 | 91 |
| | 0.030 | 27 | 34 | 39 | 42 | 45 | 50 | 54 | 59 | 63 | 68 | 72 | 74 | 78 |
| | 0.035 | 22 | 28 | 32 | 35 | 38 | 43 | 46 | 51 | 54 | 59 | 63 | 65 | 68 |
| 0.0100 | 0.009 | 130 | 151 | 162 | 170 | 175 | 185 | 191 | 199 | 204 | 210 | 213 | 217 | 222 |
| | 0.010 | 114 | 133 | 143 | 151 | 156 | 165 | 171 | 179 | 184 | 190 | 193 | 196 | 200 |
| | 0.011 | 100 | 119 | 129 | 135 | 141 | 149 | 155 | 162 | 167 | 173 | 176 | 180 | 184 |
| | 0.012 | 90 | 107 | 116 | 123 | 128 | 136 | 142 | 149 | 154 | 160 | 163 | 166 | 170 |
| | 0.013 | 81 | 98 | 106 | 112 | 117 | 125 | 130 | 138 | 142 | 148 | 151 | 154 | 159 |
| | 0.015 | 66 | 80 | 89 | 95 | 100 | 108 | 112 | 119 | 124 | 129 | 133 | 134 | 138 |
| | 0.017 | 57 | 70 | 77 | 82 | 87 | 94 | 99 | 105 | 109 | 115 | 118 | 121 | 126 |
| | 0.020 | 46 | 57 | 64 | 68 | 72 | 79 | 83 | 89 | 93 | 99 | 102 | 105 | 108 |
| | 0.025 | 34 | 44 | 49 | 53 | 56 | 62 | 66 | 71 | 76 | 81 | 84 | 86 | 90 |
| | 0.030 | 27 | 35 | 39 | 43 | 45 | 51 | 55 | 59 | 63 | 68 | 71 | 74 | 77 |
| | 0.035 | 22 | 29 | 33 | 35 | 38 | 43 | 46 | 51 | 55 | 59 | 62 | 65 | 68 |




Discharge of circular sewers for $n = 0.013$.

*Figure 35*

## 1411. MATERIALS OF CONSTRUCTION

Most of the first sewerage systems which were constructed in this country were combined systems; i.e., the sewers carried both the sanitary and storm water flow. However, the tendency in most communities at the present time, due to the expense of disposing of sanitary sewage, is toward the adoption of a separate system by which the storm water and the sanitary sewage are carried in separate sewers.

In separate sewers, the possibility of damage to the sewer from the attack of hydrogen sulphide gas is minimized in the storm sewers. However, in a great many American cities, the practice has been in the past, when the capacity of storm sewers became inadequate due to increased development, to convert these sewers to sanitary sewers and to build relief storm sewers.

In view of this practice, too great reliance should not be placed upon the fact that storm sewers are rarely subjected to attacks of hydrogen sulphide gas but reasonable provision should be made so that all sewers will withstand such attacks, and the possible disintegration of the material of which the sewer is constructed should undoubtedly

be given consideration when selecting these materials. The effects of hydrogen sulphide gas on brick sewers are discussed in Section 1405.

Another important factor, which should be given consideration in selecting materials for the construction of sewers, is the effect of abrasion upon the sewer. Regarding the erosion of sewer inverts, Metcalf and Eddy state in "American Sewerage Practice", Volume 1, page 144:

"The erosive effect of sewage upon sewer inverts of different kinds would be unimportant in the case of the separate system were it not for the fact that sand, gravel, or other silicious material does find its way into many such sewers. In the combined system, which has to deal with larger quantities of silicious material as well as with rain water and sewage, the effect may be more important. The rapidity of the erosive action will depend not only upon the velocity of flow, but also upon the character of the material transported, arenaceous material, on account of its greater hardness, being much more destructive in its influence than argillaceous or limestone. Vitrified sewer pipe is resistant to erosion and has been laid successfully upon very steep grades. In large combined sewers, it has generally been customary to line concrete or brick sewers, in the invert at least, with vitrified brick, where the velocity of flow is in excess of 8 ft. per second, although some engineers have used as low a limit as 4 ft. per second."

As previously stated, the erosive effect of sewage upon sewer inverts depends upon the quantity and quality of the silicious material which enters the sewers and the velocity of sewage flow through the sewers. It has been found that clear water flowing at high velocities (above 20 feet per second) has practically no erosive effect upon the pipes through which it flows.

In selecting the proper material, therefore, for lining the invert of a sewer, the engineer should consider not only the anticipated velocity of flow of the sewage but the quantity and character of the silicious material which may be expected to enter the sewer. In circular sewers, the design velocity (velocity flowing full) is not reached until the sewer flows half full, which means that in many sewers, particularly storm sewers, the maximum velocity will not be reached except at infrequent intervals and will be maintained then only for a relatively short time.

While it is important that adequate provision be made to prevent erosion of a sewer invert, it is manifestly poor economy and questionable engineering practice to install material at an increased cost to guard against erosive action which may never exist. Experience has demonstrated, as indicated in Section 1406, that, except under the most severe conditions, hard-burned brick of relatively high absorption will withstand the erosive action in sewers over a long period of time.

## 1412. SHAPE OF CROSS-SECTION

The proper shape which should be selected for the cross-section of a sewer will depend upon the maximum, average, and minimum sewage flow anticipated as well as conditions under which the sewer is to be constructed such as the depth of crown below the surface, character of the ground in which the sewer is to be constructed, any superimposed loads which the sewer may be required to support and other considerations.

From the standpoint of hydraulics, the circular section is the most economic for a uniform flow. However, at low rates of flow, the velocity in the circular section is materially reduced.

From the structural standpoint, the semi-elliptical section or a section consisting of three or more circular arcs, approximating the semi-ellipse, is desirable since the form of the arch coincides very closely with the line of resistance.

Due to the flat bottom of the semi-elliptical section, the velocity at low depth is small and this section cannot be used to best advantage unless the normal flow is at least one-third of the maximum flow.

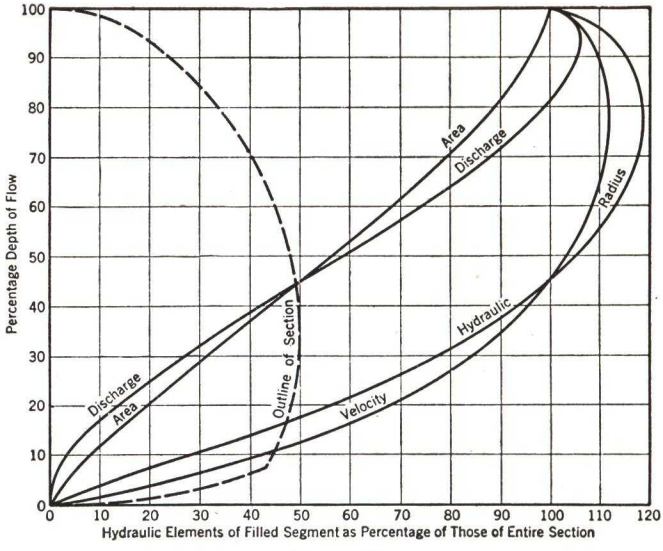



*Figure 36*

Figure 36 gives the hydraulic properties of a semi-elliptical section designed by the engineers of the Sanitary District of Chicago. The height and width of this section are the same, and the radii of the circular arcs have been so selected that the area is practically the same as the area of a circular section of the same height.

This section may be constructed economically in tunnels or in open cuts through soft material. In rock excavation, however, it may not prove economical since the excavation required is somewhat greater than for the circular type.

The structural design of a circular section and of the Chicago Sanitary District semi-elliptical section will be discussed in the succeeding pages.

While the circular or semi-elliptical section can be used to advantage in a great many instances, there are many cases where special designs will be required.

## 1413. LOADS ON SEWERS

The most comprehensive investigations which have been made to date to determine the loads transmitted to a sewer by the earth backfill and superimposed loads have been conducted during the past 25 years under the direction of Professor Anson Marston at the Engineering Experiment Station of the Iowa State College of Agriculture. Regarding these investigations, Professor Marston makes the following statement in Bulletin No. 96, "The Theory of External Loads on Closed Conduits in the Light of the Latest Experiments", published February 19th, 1930:

"Working over a period of about 21 years, and assisted at different times by several persons, the author of this paper has developed, and from time to time published, in successive stages, a complete mathematical theory of external loads on closed conduits and of the supporting strengths of pipe conduits. This theory applies to all kinds of closed conduits and to all classes of field conditions of conduit construction.

"The theory is supported strongly, and with remarkable unanimity: First, by all actual weighings on record of loads on closed conduits determined in experimental researches; second, by extensive field data obtained by several series of wide-spread examinations of actual conduits in use.

"The term 'conduits' includes culverts, drains, sewers, aqueducts, water pipes, gas mains, telephone conduits, and underground steam mains; of all shapes, materials,

degrees of rigidity, field construction conditions affecting loads, and field construction conditions affecting supporting strengths.

"As to shapes, closed conduits may be rectangular, arched, oval, circular, or of any other shape.

"As to materials, closed conduits may be of masonry (brick, plain or reinforced concrete, or stone) constructed in place or otherwise; or of pipes or conduit sections, of burnt clay, or plain or reinforced concrete, for culverts, sewers, tile drains, or telephone or other conduits; or of metal, including cast iron, steel or wrought iron, and corrugated metal; or of wood, including box sections and wood stave pipe, and bored wood pipes.

"As to degrees of rigidity, closed conduits may be classified as:

"1. Rigid conduits, whose cross sectional shapes cannot be distorted sufficiently to change their vertical or horizontal dimensions more than 0.1 per cent without causing materially injurious cracks; including all rectangular conduits, and all cylindrical conduits made of plain or reinforced concrete masonry or pipes or of burnt clay pipes.

"2. Semi-rigid conduits, whose cross sectional shapes can be distorted sufficiently to change their vertical or horizontal dimensions more than 0.1 per cent, but not more than 3.0 per cent, without causing materially injurious cracks; including segmental block conduits, and those made of cast iron pipe, together with some brick or stone block masonry cylindrical conduits.

"3. Flexible conduits, whose cross sectional shapes can be distorted sufficiently to change their vertical or horizontal dimensions more than 3.0 per cent before causing materially injurious cracks; including those made of corrugated pipe, thin steel or wrought iron pipe, and probably some cylindrical conduits made of brick or stone block masonry."

As a result of the investigations referred to above, the following formula for determining the vertical weight or load per foot of pipe, transmitted to the pipe by the earth backfill has been developed:

$$W = cwB^2$$

In this formula, W is the total load per foot of pipe, c is a coefficient depending upon the ratio of depth to width of trench and the character and condition of the backfill, w is the weight of backfilling material in pounds per cubic foot, and B is the width of the trench a little below the top of the pipe.

In the investigations of Iowa State College, an effort was made to determine the ratio of the "supporting strength" of the pipe when laid in a ditch under actual working conditions to the strength of a similar pipe when tested in a laboratory by a standard method of testing.

It was found early in the investigations that "ditch conditions" or the condition of the trench in which the pipe was laid had a decided effect upon the supporting strength of the pipe. Factors, therefore, were determined, giving the relation of "field supporting strength" to "laboratory test strength" for various ditch conditions.

It was found that for a ditch condition designated as "ordinary" in which the underside of the pipe is well-bedded in the soil for from 60 to 90 degrees of its circumference, the ratio of field supporting strength to laboratory test strength by the sand-bearing method was one (1.00). However, when the pipe is supported for all or a portion of the lower half of its circumference in a concrete cradle, its supporting strength is increased.

W. J. Schlick states as conclusion L in Iowa Engineering Experiment Station Bulletin No. 57: "The supporting strength of drain tile and sewer pipe can be increased from 50 to 100 per cent by the use of properly designed concrete cradles. . . . .

Due to the fact that the investigations of the Iowa State College Experiment Station have been concerned primarily with total loads and the ratios of total supporting strength to total test load, very little data has been reported concerning the distribution of the total load, either vertical or horizontal, upon the pipe.

Professor Marston makes the following statement regarding horizontal components of external loads in Bulletin 96, referred to above:

"Study of hundreds of measurements of actual horizontal pressures in granular fill materials, during 21 years of research, has convinced the author of the reliability of Rankine's well-known formulas, which may be stated as follows:

The Active horizontal pressure = K x (vertical pressure).

The Passive horizontal pressure not less than 1/K x (vertical pressure).

$$K = \frac{\sqrt{u^2 + 1} - u}{\sqrt{u^2 + 1} + u}$$

u = coefficient of internal friction.

"1. For 'Ditch Conduits' (Completely buried in ditches).

"The horizontal pressures against 'ditch conduits' are wholly dependent on the class of bedding and are best taken into account by using conduit supporting strength bedding ratios of field supporting strength to laboratory test strength.

"2. For 'Projecting Conduits' (Under embankments and projecting above the embankment sub-grades).

"(a) For rigid conduits, not injuriously cracked: Active horizontal pressure equal to K x (vertical pressure) should be assumed against the projecting conduit sides.

"(b) For semi-rigid conduits: Slightly greater horizontal pressures should be assumed than for rigid conduits.

"(c) For flexible conduits, and for all badly cracked conduits: The development of passive resistance to elongation of the horizontal diameter will often cause horizontal pressures equal to the vertical pressures."

In Iowa Engineering Experiment Station Bulletin No. 112, "The Supporting Strength of Rigid Pipe Culverts", published February 8th, 1933, Mr. M. G. Spangler states:

"Marston's theory of loads on conduits due to earth fills has been substantiated by the results of extensive research previously conducted here and elsewhere. It is used as a major premise in the theoretical phase of this study. The substantial agreement between the theoretical and experimental values of the load factor provides further evidence of the reliability of Marston's theory.

"Marston's theory assumes the vertical load on a conduit to be uniformly distributed over its breadth. An analysis of the measurements of the radial pressure on two culverts in this study seems to verify this assumption.

"It was stated as conclusion No. 1 in Bulletin 76 (16) 'that it is safe to assume that active horizontal pressures about equal to those calculated by Rankine's formula may be considered to act against those portions of (rigid) pipe culverts which project above the surface of the natural ground adjacent.' This conclusion was based upon observations and experience obtained in many years of experimental work with loads on conduits due to ditch filling and embankment materials previous to the issuance of that bulletin (1926). It is here reaffirmed, as the experimental work since that date has, in a number of ways, tended to strengthen the conclusion, and safe working values of the load factor have been calculated on this basis.

"It is not intended to imply, however, that the portion of the culvert which projects above the surface of the natural ground adjacent is the maximum area upon which the active lateral pressure may act in all cases. It seems possible that under certain conditions the lateral pressure might act over the entire sides of the pipe regardless of the projection ratio. To illustrate, when a rigid pipe culvert is installed with little or no projection ratio; that is, in a ditch whose depth is nearly equal to the vertical dimension of the pipe, the lateral pressure on the pipe will be negligible when the natural ground in which the ditch is dug is solid rock or compact, unyielding earth, since these materials

are incapable of producing or transmitting active pressures in their natural state. But when the ditch is dug in a yielding earth, and when the embankment is of considerable height, the vertical load on the natural ground surface adjacent to the culvert may become great enough to break down the earth and destroy its internal cohesion to such an extent that it may flow laterally and exert an active pressure against the sides of the pipe. If this action occurs, the load factor ceases to be a function of the projection ratio."

The fact that the field supporting strength of pipe laid under "ordinary" ditch conditions is the same as the laboratory sand-bearing test strength and that the addition of a concrete cradle increases the field supporting strength from 50 to 100 per cent above the laboratory sand-bearing test strength would seem to indicate that the load distribution on the lower half of the circumference of the pipe under ordinary ditch conditions causes the critical stresses in the pipe and that the load distribution on the upper half of the circumference is more favorable insofar as the supporting strength of the pipe is concerned, since with a more uniform distribution of the forces acting upon the lower half of the circumference through the use of a concrete cradle, the supporting strength of the pipe is materially increased.

A brick sewer constructed in a trench is not comparable in all respects to any of the pipe tested at the Iowa Experiment Station. However, since the lower half of the circumference of a brick sewer is fully supported, the ditch condition which should be considered for a brick sewer in most cases would be comparable to pipes supported in concrete cradles.

In the design of an arch for a brick sewer, it is necessary to know not only the total vertical and horizontal loads but the distribution of these loads. The following statements are taken from Baker's "Masonry Construction", Tenth Edition, page 616:

"We know certainly that the passive resistance of the earth adds materially to the stability of masonry arches; for the arch rings of many sewers which stand without any evidence of weakness are in a state of unstable equilibrium, if the vertical pressure of the earth immediately above the ring be considered as the only external force acting upon it."

Rankine's formula for earth pressure is then given, after which it is stated: "The condition in which the earth will be deposited behind the arch cannot be foretold; but it is probable that at least the minimum value, as above, will always be realized. Hence we will assume that the horizontal intensity is at least one-third of the vertical intensity. . . . And again on page 638, it is stated: "Although the horizontal components of the external forces cannot be accurately determined, any theory that disregards them is needlessly inaccurate."

In the design of brick sewers, some engineers consider only the vertical forces acting upon the ring of the sewer, either the total weight of the earth above the sewer or the total vertical load, as determined by Marston's formula, and neglect entirely the horizontal components or horizontal pressures. In the opinion of the author and, as stated by Professor Baker, this method seems needlessly inaccurate, and while it is true that the exact amount of the horizontal pressure cannot be determined, there is overwhelming evidence, obtained both from experiments and from structures which have been constructed and are now in service, that a substantial horizontal pressure does exist.

In the light of present available data, it seems probable that this pressure is greater for the larger diameter sewers (6 ft. and over) than for the smaller sewers. The designs of brick sewers presented herein are based, therefore, upon horizontal pressures equal to one-third of the vertical pressures. Except under the most severe conditions, which will require special study, it is believed that sewers designed upon this basis will be safe and will give satisfactory service.

The following tables 49 and 50, taken from Iowa Engineering Experiment Station Bulletin No. 31, give values, as recommended by Professor Marston, of c and w for use in the equation, $W = cwB^2$.

## TABLE NO. 49

## APPROXIMATE SAFE WORKING VALUES OF c IN THE MARSTON AND ANDERSON TRENCH-PRESSURE FORMULA, $W = cwB^2$

| Ratio of depth to width* | Values of c for— | | | |
|---|---|---|---|---|
| | Damp top soil and dry and wet sand | Saturated top soil | Damp yellow clay | Saturated yellow clay |
| 0.5 | 0.46 | 0.47 | 0.47 | 0.48 |
| 1.0 | 0.85 | 0.86 | 0.88 | 0.90 |
| 1.5 | 1.18 | 1.21 | 1.25 | 1.27 |
| 2.0 | 1.47 | 1.51 | 1.56 | 1.62 |
| 2.5 | 1.70 | 1.77 | 1.83 | 1.91 |
| 3.0 | 1.90 | 1.99 | 2.08 | 2.19 |
| 3.5 | 2.08 | 2.18 | 2.28 | 2.43 |
| 4.0 | 2.22 | 2.35 | 2.47 | 2.65 |
| 4.5 | 2.34 | 2.49 | 2.63 | 2.85 |
| 5.0 | 2.45 | 2.61 | 2.78 | 3.02 |
| 5.5 | 2.54 | 2.72 | 2.90 | 3.18 |
| 6.0 | 2.61 | 2.81 | 3.01 | 3.32 |
| 6.5 | 2.68 | 2.89 | 3.11 | 3.44 |
| 7.0 | 2.73 | 2.95 | 3.19 | 3.55 |
| 7.5 | 2.78 | 3.01 | 3.27 | 3.65 |
| 8.0 | 2.82 | 3.06 | 3.33 | 3.74 |
| 8.5 | 2.85 | 3.10 | 3.39 | 3.82 |
| 9.0 | 2.88 | 3.14 | 3.44 | 3.89 |
| 9.5 | 2.90 | 3.18 | 3.48 | 3.96 |
| 10.0 | 2.92 | 3.20 | 3.52 | 4.01 |
| 11.0 | 2.95 | 3.25 | 3.58 | 4.11 |
| 12.0 | 2.97 | 3.28 | 3.63 | 4.19 |
| 13.0 | 2.99 | 3.31 | 3.67 | 4.25 |
| 14.0 | 3.00 | 3.33 | 3.70 | 4.30 |
| 15.0 | 3.01 | 3.34 | 3.72 | 4.34 |

*The depth of trench is to the top of the pipe.

## TABLE NO. 50

## PROPERTIES OF DITCH-FILLING MATERIALS

### Marston and Anderson

| Material | Weight of filling, lb. per cu. ft. | Ratio of lateral to vertical earth pressures | Coefficient of friction against sides of trench | Coefficient of internal friction |
|---|---|---|---|---|
| Partly compacted damp top soil..... | 90 | 0.33 | 0.50 | 0.53 |
| Saturated top soil..... | 110 | 0.37 | 0.40 | 0.47 |
| Partly compacted damp yellow clay.. | 100 | 0.33 | 0.40 | 0.52 |
| Saturated yellow clay..... | 130 | 0.37 | 0.30 | 0.47 |
| Dry sand..... | 100 | 0.33 | 0.50 | 0.55 |
| Wet sand..... | 120 | 0.33 | 0.50 | 0.57 |

## TABLE NO. 51

## APPROXIMATE MAXIMUM LOADS, IN POUNDS PER LINEAR FOOT, ON PIPE IN TRENCHES, IMPOSED BY COMMON FILLING MATERIALS

### Marston and Anderson

| Depth of fill above pipe | Breadth of ditch at top of pipe | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1 ft. | 2 ft. | 3 ft. | 4 ft. | 5 ft. | 1 ft. | 2 ft. | 3 ft. | 4 ft. | 5 ft. |
| | Partly compacted damp top soil; 90 lbs. per cubic foot | | | | | Saturated top soil; 110 lbs. per cubic foot | | | | |
| 2 ft. | 130 | 310 | 490 | 670 | 830 | 170 | 380 | 600 | 820 | 1,020 |
| 4 ft. | 200 | 530 | 880 | 1,230 | 1,580 | 260 | 670 | 1,090 | 1,510 | 1,950 |
| 6 ft. | 230 | 690 | 1,190 | 1,700 | 2,230 | 310 | 870 | 1,500 | 2,140 | 2,780 |
| 8 ft. | 250 | 800 | 1,430 | 2,120 | 2,790 | 340 | 1,030 | 1,830 | 2,660 | 3,510 |
| 10 ft. | 260 | 880 | 1,640 | 2,450 | 3,290 | 350 | 1,150 | 2,100 | 3,120 | 4,150 |
| | Dry sand; 100 lbs. per cubic foot | | | | | *Saturated sand; 120 lbs. per cubic foot | | | | |
| 2 ft. | 150 | 340 | 550 | 740 | 930 | 180 | 410 | 650 | 890 | 1,110 |
| 4 ft. | 220 | 590 | 970 | 1,360 | 1,750 | 270 | 710 | 1,170 | 1,640 | 2,100 |
| 6 ft. | 260 | 760 | 1,320 | 1,890 | 2,480 | 310 | 910 | 1,590 | 2,270 | 2,970 |
| 8 ft. | 280 | 890 | 1,590 | 2,350 | 3,100 | 340 | 1,070 | 1,910 | 2,820 | 3,720 |
| 10 ft. | 290 | 980 | 1,820 | 2,720 | 3,650 | 350 | 1,180 | 2,180 | 3,260 | 4,380 |
| 12 ft. | 300 | 1,040 | 2,000 | 3,050 | 4,150 | 360 | 1,250 | 2,400 | 3,650 | 4,980 |
| 14 ft. | 300 | 1,090 | 2,140 | 3,320 | 4,580 | 360 | 1,310 | 2,570 | 3,990 | 5,490 |
| 16 ft. | 300 | 1,130 | 2,260 | 3,550 | 4,950 | 360 | 1,350 | 2,710 | 4,260 | 5,940 |
| 18 ft. | 300 | 1,150 | 2,350 | 3,740 | 5,280 | 360 | 1,380 | 2,820 | 4,490 | 6,330 |
| 20 ft. | 300 | 1,170 | 2,420 | 3,920 | 5,550 | 360 | 1,400 | 2,910 | 4,700 | 6,660 |
| 22 ft. | 300 | 1,180 | 2,480 | 4,060 | 5,800 | 360 | 1,420 | 2,980 | 4,880 | 6,960 |
| 24 ft. | 300 | 1,190 | 2,540 | 4,180 | 6,030 | 360 | 1,430 | 3,050 | 5,010 | 7,230 |
| 26 ft. | 300 | 1,200 | 2,570 | 4,290 | 6,210 | 360 | 1,440 | 3,090 | 5,150 | 7,460 |
| 28 ft. | 300 | 1,200 | 2,600 | 4,370 | 6,390 | 360 | 1,440 | 3,120 | 5,240 | 7,670 |
| 30 ft. | 300 | 1,200 | 2,630 | 4,450 | 6,530 | 360 | 1,440 | 3,150 | 5,340 | 7,830 |
| | Partly compacted damp yellow clay; 100 lbs. per cubic foot | | | | | *Saturated yellow clay; 130 lbs. per cubic foot | | | | |
| 2 ft. | 160 | 350 | 550 | 750 | 930 | 210 | 470 | 730 | 1,000 | 1,240 |
| 4 ft. | 250 | 620 | 1,010 | 1,400 | 1,800 | 340 | 840 | 1,330 | 1,870 | 2,370 |
| 6 ft. | 300 | 830 | 1,400 | 1,990 | 2,580 | 430 | 1,140 | 1,900 | 2,630 | 3,410 |
| 8 ft. | 330 | 990 | 1,720 | 2,500 | 3,250 | 490 | 1,380 | 2,360 | 3,360 | 4,400 |
| 10 ft. | 350 | 1,110 | 2,000 | 2,920 | 3,880 | 520 | 1,570 | 2,760 | 3,980 | 5,270 |
| 12 ft. | 360 | 1,200 | 2,220 | 3,320 | 4,450 | 540 | 1,730 | 3,100 | 4,560 | 6,050 |
| 14 ft. | 370 | 1,280 | 2,410 | 3,650 | 4,950 | 560 | 1,850 | 3,410 | 5,050 | 6,760 |
| 16 ft. | 370 | 1,330 | 2,570 | 3,950 | 5,400 | 570 | 1,940 | 3,660 | 5,510 | 7,440 |
| 18 ft. | 380 | 1,380 | 2,710 | 4,210 | 5,810 | 570 | 2,020 | 3,880 | 5,930 | 8,060 |
| 20 ft. | 380 | 1,410 | 2,830 | 4,450 | 6,180 | 580 | 2,090 | 4,070 | 6,280 | 8,610 |
| 22 ft. | 380 | 1,430 | 2,920 | 4,640 | 6,500 | 580 | 2,140 | 4,240 | 6,610 | 9,130 |
| 24 ft. | 380 | 1,450 | 3,000 | 4,820 | 6,800 | 580 | 2,180 | 4,380 | 6,910 | 9,590 |
| 26 ft. | 380 | 1,470 | 3,060 | 4,980 | 7,080 | 580 | 2,210 | 4,500 | 7,180 | 10,010 |
| 28 ft. | 380 | 1,480 | 3,120 | 5,100 | 7,310 | 580 | 2,240 | 4,610 | 7,380 | 10,430 |
| 30 ft. | 380 | 1,490 | 3,170 | 5,230 | 7,530 | 580 | 2,260 | 4,700 | 7,590 | 10,780 |

*These two subtables contain the most important figures for practical use.

Table No. 51 gives the value of W (total load) on pipes in trenches of various widths and depths.

Table No. 52 gives the fractional part of "long" superimposed loads which are transmitted to pipes in trenches of various depths and widths. "Long" loads, as used in this table, are loads which extend along the trench for a considerable distance as compared to the width of the trench such as piles of lumber or building materials. Values given in this table have been verified by test.

For the fractional part of "short" superimposed loads which are transmitted to pipes, the reader is referred to Iowa Experiment Station Bulletin No. 79, "Experimental Determination of Static & Impact Loads Transmitted to Culverts".

**TABLE NO. 52**

**PROPORTION OF "LONG" SUPERFICIAL LOADS ON BACKFILLING WHICH REACHES THE PIPE IN TRENCHES WITH DIFFERENT RATIOS OF DEPTH TO WIDTH AT TOP OF PIPE**

**Marston and Anderson**

| Ratio of depth to width | Sand and damp top soil | Saturated top soil | Damp yellow clay | Saturated yellow clay |
|---|---|---|---|---|
| 0.0 | 1.00 | 1.00 | 1.00 | 1.00 |
| 0.5 | 0.85 | 0.86 | 0.88 | 0.89 |
| 1.0 | 0.72 | 0.75 | 0.77 | 0.80 |
| 1.5 | 0.61 | 0.64 | 0.67 | 0.72 |
| 2.0 | 0.52 | 0.55 | 0.59 | 0.64 |
| 2.5 | 0.44 | 0.48 | 0.52 | 0.57 |
| 3.0 | 0.37 | 0.41 | 0.45 | 0.51 |
| 4.0 | 0.27 | 0.31 | 0.35 | 0.41 |
| 5.0 | 0.19 | 0.23 | 0.27 | 0.33 |
| 6.0 | 0.14 | 0.17 | 0.20 | 0.26 |
| 8.0 | 0.07 | 0.09 | 0.12 | 0.17 |
| 10.0 | 0.04 | 0.05 | 0.07 | 0.11 |

## 1414. THEORY OF DESIGN

Regarding the design of masonry arches, Professor Baker states in "Masonry Construction", page 608:

"There are two classes of theories of the stability of the masonry arch — the line of thrust theories and the elastic deformation theories. The line of thrust theory considers the stability of the arch ring as depending upon the friction and the reactions between the several arch stones; while the elastic theory regards the arch as a curved beam which depends for its stability upon the internal stresses developed in the material of the arch. Both theories can be applied to either a voussoir or a continuous arch, although usually the line of thrust theory is employed for the voussoir arch, and the elastic theory for the monolithic arch. There is no great difference between the two theories, although the elastic theory is a little more complicated but a little more accurate."

As stated by Professor Baker, the "line of thrust" theories are usually applied to plain masonry arches and the elastic theories to reinforced masonry arches. In the following designs, therefore, the line of thrust or, as it is sometimes called, the static method will be used in the design of the 10 ft. circular sewer of plain brick masonry and

the elastic theory will be used in the design of the semi-elliptical reinforced brick masonry sewer.

All theories for the design of masonry arches are really methods of verification. The dimensions of the arch are first assumed, based upon common practice or an empirical formula, and the assumed arch is then tested by one or more of the theories. If the line of resistance, as determined by the theory, does not lie within the prescribed limits, the dimensions of the arch are altered and the design is tested again.

## 1415. DESIGN OF UNREINFORCED BRICK SEWER

**(a) Assumptions:**

1. Sewer to be constructed in unyielding soil such as rock, shale, or hard clay.
2. Depth from surface of ground to top of sewer (D) is 24 ft.
3. Weight of backfilling material, 100 pounds per cubic foot (partly compacted damp yellow clay).
4. Horizontal earth pressure equals ⅓ of vertical pressure.
5. Vertical pressure equals that obtained from Marston's formula, $W = cwB^2$.
6. Circular section, 10 ft. diameter.

The assumption that this sewer is to be constructed in unyielding soil would apply in the case of a large percentage of sewers of this size which are actually constructed. Large sewers are usually trunk sewers and must, therefore, be constructed at depths where the soil conditions assumed above are frequently encountered.

In unyielding soil, the circular arch can be used to advantage since the sides of the trench can be relied upon to withstand the horizontal thrust at the spring line of the arch and in such cases it is rarely necessary to include reinforcing steel in the arch.

In selecting a preliminary design for a sewer arch, empirical formulas or a study of existing sewers, is usually relied upon. With slight variations of the diameters at which the thickness of the arch is increased by one ring of brick masonry, the following represents what is probably a fair average of common practices in brick sewer construction.

Sewers up to 6 ft. inside diameter — 2 brick thick.
Sewers up to 10 ft. — 3 brick thick.
Sewers above 10 ft. — 4 brick thick.

In accordance with the above, a three ring brick arch will be assumed for the 10 ft. diameter sewer under consideration.

**(b) Analysis of three-ring sewer:**

The theory by which the stresses in this arch will be analyzed is known as the rational theory and is based upon the hypothesis of least crown thrust. This theory is described in detail in Baker's "Masonry Construction", Tenth Edition, page 628.

The hypothesis of least crown thrust is, that the true line of resistance for the arch is that for which the thrust at the crown is the least possible consistent with equilibrium. This theory was first proposed by Moseley in 1837.

The first step in the analysis is to determine the joint of rupture. The joint of rupture is that joint for which the tendency of the arch to open at the extrados is the greatest and which, therefore, requires the greatest crown thrust applied at the upper extremity of the middle third to prevent the joint from opening.

In order to determine the joint of rupture, the arch is divided into segments commonly called voussoirs. If the length of these segments is relatively small, the equilibrium polygon constructed on the arch will approach the true line of resistance. Actually, the true line of resistance is a continuous curve circumscribed by the equilibrium polygon.

In this analysis, the length of the segments of voussoirs is taken at approximately one foot.

After the arch has been divided into voussoirs, the amount and point of application of the external forces, acting upon each voussoir, must be determined. In this instance, we have only the vertical and horizontal components of the earth pressure.

To obtain the vertical pressure, the following values should be substituted in Marston's formula:

$B = 12$ feet
$w = 100$ pounds per cubic foot
$D = 24$ feet
$\frac{D}{H} = 2$
$c = 1.56$ (Table No. 49)
$W = cwB^2 = 22{,}500$ pounds per linear foot.

If we assume that the vertical pressure is distributed uniformly over the horizontal projection of the sewer, the vertical component of the earth pressure equals 22,500 divided by 12 equals 1875 pounds per square foot. To this will be added the weight of the voussoirs, assumed as 140 pounds each. Since the weights of the voussoirs are small as compared to the vertical component of the earth pressure, the total vertical force will be assumed as acting at the center of the horizontal projection of each voussoir.

Assuming that the horizontal pressure equals $\frac{1}{3}$ the vertical pressure, h (horizontal pressure) equals 625 pounds per square foot. This force will be assumed as acting at the center of the vertical projection of each voussoir. This assumption is practically correct, since the vertical projection of the voussoirs is small as compared to the total depth of the sewer.

The horizontal and vertical projection of the voussoirs of the arch, illustrated on Figure 37, are listed in columns 19 and 21 respectively of Table No. 53, and the vertical and horizontal forces acting on each voussoir are listed in columns 20 and 22 respectively.

In this analysis, the loading on the arch is assumed as symmetrical about the center line and only half of the arch, therefore, is considered. This section from the spring line to the crown is held in equilibrium by the external forces, the reaction at the skewback and the horizontal thrust at the crown.

The next step in the determination of joint of rupture is to take moments of the external forces acting upon the arch between the joint in question and the crown about the lower extremity of the middle third of the joint.

The moment arms of the external forces measured from the lower extremity of the middle third of each joint are listed in columns 2 to 17 of Table No. 53. In this table, opposite Joint 2, x1 represents the horizontal distance of the vertical force acting on voussoir 1 from the lower third point of Joint 2. x2 represents the horizontal distance of the vertical force acting on voussoir 2 from the same point. Similarly, K1 represents the vertical distance of the horizontal force acting on voussoir 1 from the point of moments, and K2 represents the vertical distance of the horizontal force acting on voussoir 2 from the point of moments.

The values of x and K, listed opposite the other joints, represent the moment arms of the various forces measured from the lower third points of these joints.

The values of y, listed in column 18, are the moment arms of the crown thrust from the lower third point of each joint. $\Sigma$ wx, column 23, is the sum of the moments of the vertical forces and $\Sigma$ hK, column 24, is the sum of the moments of the horizontal forces.

Column 25 gives values of $H_o$, required to keep each joint from opening at extrados, and the maximum value in this column represents the true crown thrust. The joint for which this maximum value is obtained is the joint of rupture.

For the arch assumed, it will be noted (Table 53, column 25) that the maximum crown thrust for Joint 5, which is the joint of rupture, is only 46 pounds greater than

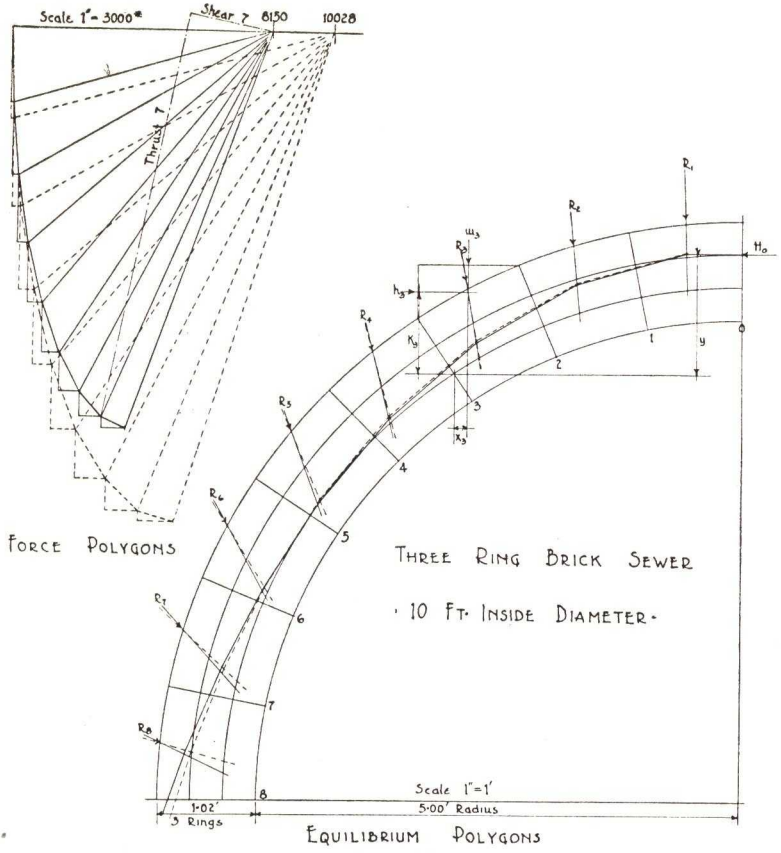



*Figure 37*

the required crown thrust for Joint 4. If a joint had been assumed between Joint 4 and Joint 5, a slightly greater crown thrust would probably have been obtained for this joint, indicating that the true joint of rupture falls between Joint 4 and Joint 5. This is also indicated by the equilibrium polygon, Figure 37.

Table 54 is similar to Table 53 except that moments are taken about the upper third of each joint and the crown thrust obtained, column 25, is the maximum thrust which may be applied before the joint opens at the intrados.

From this table, the true skewback and spring line of the arch may be determined. Since $H_o$ from Table 54 for Joint 8 is less than $H_o$ from Table 53 for Joint 5, the skewback of the arch lies somewhere between Joint 7 and Joint 8 and the equilibrium polygon will pass out of the middle third of the arch somewhere between Joint 7 and Joint 8.

After the true crown thrust has been determined, a force polygon is constructed, Figure 37, and the equilibrium polygon is then laid off on the arch. The equilibrium polygon must pass through the lower third point of the joint of rupture which serves as a check upon the accuracy of the calculations.

It will be noted from Figure 37 that the equilibrium polygon is slightly outside of the middle third between Joints 4 and 5, indicating that the true joint of rupture lies between these points.

## TABLE NO. 53

## COMPUTATION OF MOMENTS AND CROWN THRUST — STATIC METHOD — FOR 10 FOOT INSIDE DIAMETER 3 RING BRICK SEWER

**Moments about lower third point of each joint**

(Columns 20 to 24 inc. V load 1875 lb./sq. ft. (Marston's formula); Columns 26 to 30 inc. V load 2400 lb./sq. ft. (total weight of earth above sewer)

COLUMNS

| 1 Jt. | 2 $x_1$ | 3 $x_2$ | 4 $x_3$ | 5 $x_4$ | 6 $x_5$ | 7 $x_6$ | 8 $x_7$ | 9 $x_8$ | 10 $K_1$ | 11 $K_2$ | 12 $K_3$ | 13 $K_4$ | 14 $K_5$ | 15 $K_6$ | 16 $K_7$ | 17 $K_8$ | 18 y | 19 H. Proj | 20 w | 21 V. Proj | 22 h | 23 $\Sigma$ wx | 24 $\Sigma$ hK | 25 $H_o$ | 26 w | 27 h | 28 $\Sigma$ wx | 29 $\Sigma$ hK | 30 $H_o$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | .45 | | | | | | | | .72 | | | | | | | | .45 | 1.17 | 2334 | .11 | 69 | 1050 | 50 | 2450 | 2815 | 88 | 1267 | 63 | 2955 |
| 2 | 1.45 | .30 | | | | | | | 1.03 | .81 | | | | | | | .75 | 1.13 | 2259 | .34 | 213 | 4062 | 244 | 5750 | 2745 | 275 | 4905 | 314 | 6958 |
| 3 | 2.38 | 1.23 | .14 | | | | | | 1.53 | 1.30 | .85 | | | | | | 1.25 | 1.04 | 2090 | .56 | 350 | 8627 | 679 | 7440 | 2572 | 462 | 10436 | 885 | 9056 |
| 4 | 3.19 | 2.04 | .95 | −.03 | | | | | 2.19 | 1.96 | 1.51 | .85 | | | | | 1.91 | .92 | 1865 | .75 | 469 | 13983 | 1496 | 8104 | 2335 | 634 | 16953 | 1969 | 9906 |
| 5 | 3.85 | 2.70 | 1.62 | .64 | −.19 | | | | 3.00 | 2.77 | 2.33 | 1.67 | .84 | | | | 2.73 | .75 | 1546 | .92 | 575 | 19371 | 2879 | 8150 | 1966 | 803 | 23537 | 3836 | 10028 |
| 6 | 4.34 | 3.20 | 2.11 | 1.14 | .3 | −.34 | | | 3.92 | 3.69 | 3.24 | 2.58 | 1.76 | .78 | | | 3.65 | .56 | 1190 | 1.04 | 650 | 23954 | 4919 | 7920 | 1522 | 942 | 29163 | 6641 | 9809 |
| 7 | 4.64 | 3.50 | 2.41 | 1.43 | .6 | −.04 | −.5 | | 4.92 | 4.69 | 4.24 | 3.58 | 2.76 | 1.78 | .70 | | 4.65 | .34 | 778 | 1.13 | 706 | 26932 | 7739 | 7430 | 961 | 1027 | 32844 | 10564 | 9313 |
| 8 | 4.75 | 3.60 | 2.51 | 1.53 | .7 | .06 | −.4 | −.62 | 5.96 | 5.73 | 5.28 | 4.62 | 3.78 | 2.82 | 1.74 | .58 | 5.68 | .11 | 346 | 1.17 | 731 | 27945 | 11305 | 6900 | 323 | 1147 | 34165 | 15611 | 8763 |

All dimensions and distances are in feet.

## TABLE NO. 54

## COMPUTATION OF MOMENTS AND CROWN THRUST — STATIC METHOD — FOR 10 FOOT INSIDE DIAMETER 3 RING BRICK SEWER

### Moments about upper third point of each joint

Columns 20 to 24 inc. V load 1875 lb./sq. ft. (Marston's formula); Columns 26 to 30 inc. V load 2400 lb./sq. ft. (total weight of earth above sewer)

| | COLUMNS | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 Jt. | 2 $x_1$ | 3 $x_2$ | 4 $x_3$ | 5 $x_4$ | 6 $x_5$ | 7 $x_6$ | 8 $x_7$ | 9 $x_8$ | 10 $K_1$ | 11 $K_2$ | 12 $K_3$ | 13 $K_4$ | 14 $K_5$ | 15 $K_6$ | 16 $K_7$ | 17 $K_8$ | 18 y | 19 H-Proj | 20 w | 21 V-Proj | 22 h | 23 Σ wx | 24 Σ hK | 25 $H_o$ | 26 w | 27 h | 28 Σ wx | 29 Σ hK | 30 $H_o$ |
| 1 | .52 | | | | | | | | .39 | | | | | | | | .11 | 1.17 | 2334 | .11 | 69 | 1214 | 27 | 11287 | 2815 | 88 | 1464 | 34 | 13618 |
| 2 | 1.59 | .44 | | | | | | | .72 | .49 | | | | | | | .43 | 1.13 | 2259 | .34 | 213 | 4705 | 154 | 11300 | 2745 | 275 | 5684 | 198 | 13679 |
| 3 | 2.57 | 1.42 | .34 | | | | | | 1.24 | 1.02 | .56 | | | | | | .96 | 1.04 | 2090 | .56 | 350 | 9917 | 499 | 10850 | 2572 | 462 | 12006 | 648 | 13181 |
| 4 | 3.43 | 2.28 | 1.20 | .22 | | | | | 1.95 | 1.73 | 1.27 | .61 | | | | | 1.67 | .92 | 1865 | .75 | 469 | 16074 | 1233 | 10363 | 2335 | 634 | 19514 | 1622 | 12656 |
| 5 | 4.14 | 2.99 | 1.91 | .93 | .10 | | | | 2.81 | 2.59 | 2.13 | 1.47 | .64 | | | | 2.53 | .75 | 1546 | .92 | 575 | 22298 | 2548 | 9820 | 1966 | 803 | 27142 | 3389 | 12067 |
| 6 | 4.67 | 3.52 | 2.44 | 1.45 | .62 | −.03 | | | 3.79 | 3.57 | 3.11 | 2.45 | 1.63 | .65 | | | 3.51 | .56 | 1190 | 1.04 | 650 | 27578 | 4959 | 9270 | 1522 | 942 | 33643 | 6227 | 11359 |
| 7 | 4.99 | 3.84 | 2.76 | 1.78 | .74 | .29 | −.16 | | 4.85 | 4.63 | 4.17 | 3.52 | 2.69 | 1.72 | .63 | | 4.58 | .34 | 778 | 1.13 | 706 | 30736 | 7541 | 8357 | 961 | 1027 | 37585 | 10285 | 10452 |
| 8 | 5.09 | 3.94 | 2.86 | 1.88 | 1.05 | .40 | −.05 | −.28 | 5.96 | 5.74 | 5.28 | 4.62 | 3.79 | 2.82 | 1.73 | .58 | 5.68 | .11 | 346 | 1.17 | 731 | 32265 | 11306 | 7671 | 323 | 1147 | 39424 | 15611 | 9689 |

All dimensions and distances are in feet.

## TABLE NO. 55

### SHEARING AND COMPRESSIVE STRESSES
### 10 FOOT INSIDE DIAMETER 3 RING BRICK SEWER

Vertical load 1875 lb./sq. ft. (Marston's formula)

| 1<br>Jt. | 2<br>V — lbs. | 3<br>T — lbs. | 4<br>d — ft. | 5<br>v #/sq. in. | 6<br>c #/sq. in. |
|---|---|---|---|---|---|
| 1 | 725 | 8320 | .13 | 5.03 | 102.84 |
| 2 | 1250 | 9000 | .03 | 8.68 | 73.75 |
| 3 | 1400 | 9950 | .08 | 9.72 | 102.26 |
| 4 | 1050 | 11000 | .16 | 7.29 | 149.72 |
| 5 | 250 | 11950 | .17 | 1.73 | 167.63 |
| 6 | 1000 | 12650 | .09 | 6.94 | 135.28 |
| 7 | 2620 | 12820 | .07 | 18.19 | 121.09 |

## TABLE NO. 56

### SHEARING AND COMPRESSIVE STRESSES
### 10 FOOT INSIDE DIAMETER 3 RING BRICK SEWER

Vertical load 2400 lb./sq. ft. (total weight of earth above sewer)

| 1<br>Jt. | 2<br>V — lbs. | 3<br>T — lbs. | 4<br>d — ft. | 5<br>v #/sq. in. | 6<br>c#/sq. in. |
|---|---|---|---|---|---|
| 1 | 850 | 10240 | .14 | 5.90 | 130.84 |
| 2 | 1420 | 11060 | .05 | 9.86 | 99.84 |
| 3 | 1650 | 12160 | .07 | 11.45 | 119.91 |
| 4 | 1350 | 13480 | .14 | 9.37 | 172.24 |
| 5 | 450 | 14700 | .17 | 3.12 | 206.20 |
| 6 | 940 | 15520 | .11 | 6.52 | 178.91 |

All dimensions and distances are in feet.

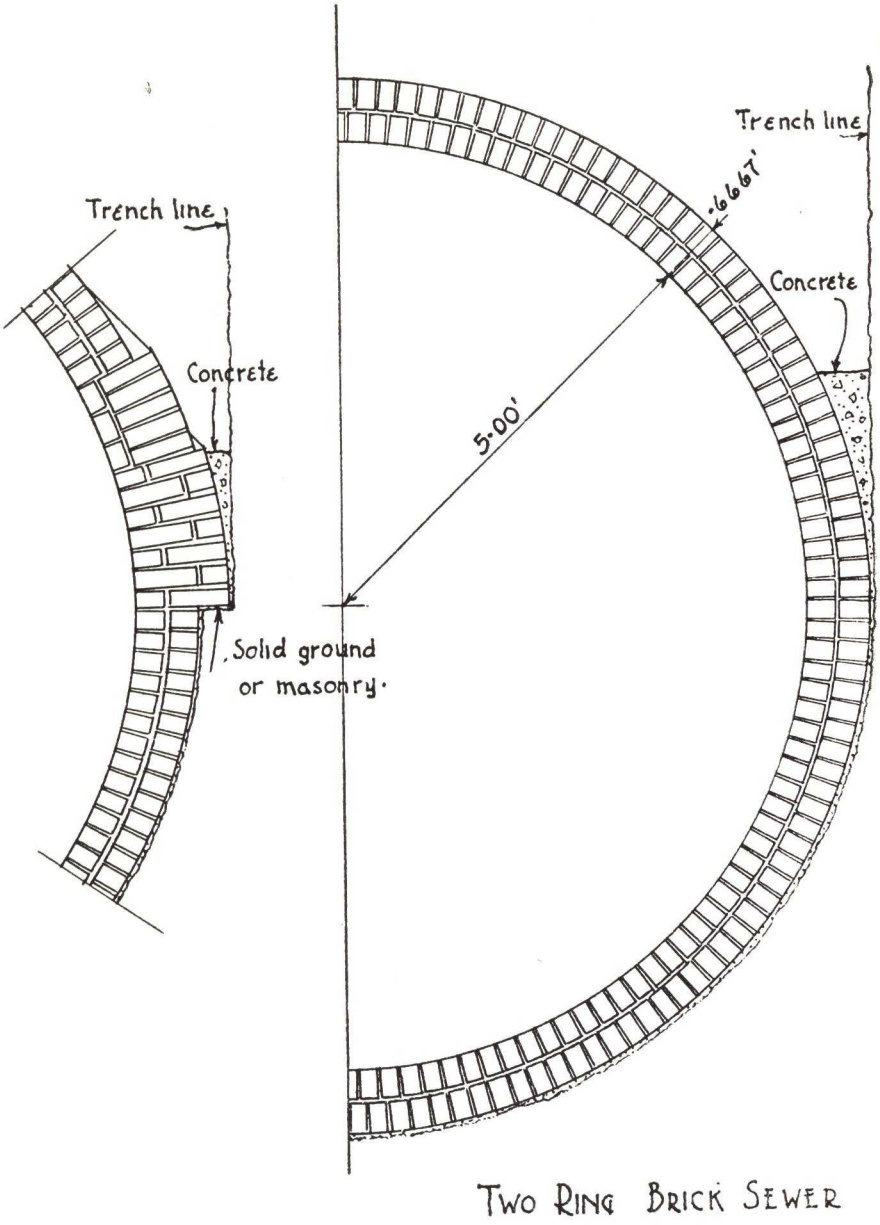



*Figure 38*

*Sections of brick sewers, illustrating methods of providing additional masonry to resist the arch thrust.*

Columns 26 and 27 of Table No. 53 represent the vertical and horizontal forces acting upon each voussoir if the total weight of the backfilling is assumed as the vertical force acting on the sewer and ⅓ of this force is assumed as the horizontal force. It will be noted that the joint of rupture falls at the same joint as for the loading, using the Marston formula for vertical pressure. The force and equilibrium polygons have also been constructed for this loading and are indicated on Figure 37 in dotted lines.

If the equilibrium polygon or line of resistance of an arch lies within the middle third, the arch is in equilibrium and the compressive and shearing stresses in the masonry may be determined by resolving the resultant force acting upon each joint into components perpendicular and parallel to the joint.

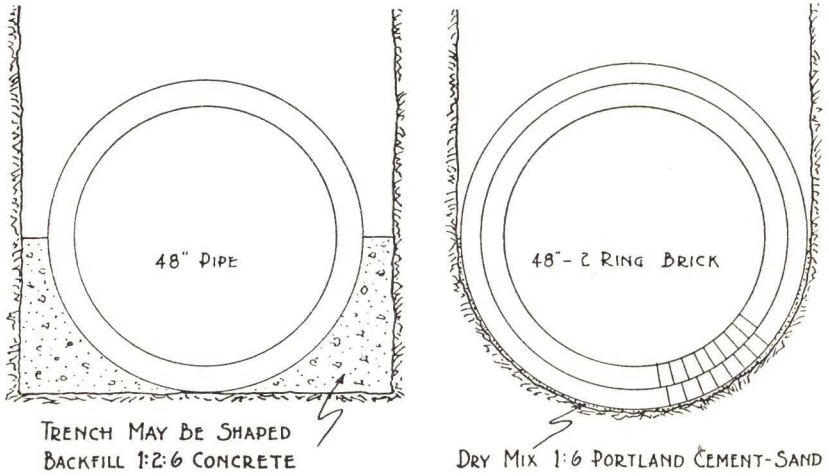
The maximum unit compressive stress may be calculated from the formula, $P = \frac{W}{l} + \frac{6Wd}{l^2}$, in which P is the maximum pressure on the joint per unit of area; W is the total normal pressure on the joint per unit of length of the arch; 1 is the depth of the joint; i.e., the distance from intrados to extrados; and d is the distance from the center of pressure to the middle of the joint (see Baker's "Masonry Construction", page 611, for derivation of this formula).

The unit shearing stress is obtained by dividing the total shear by the area of the joint.

The unit shearing and compressive stresses at each joint of the three ring brick sewer are tabulated in Table No. 55. In this table, the values listed in columns 2 and 3 are the total shear and thrust respectively, acting on each joint, obtained by resolving the resultant force into its components as indicated above. Column 4 is the eccentricity or the distance from the center of the joint to the point of application of the thrust. Columns 5 and 6 are unit shearing and compressive stresses respectively in pounds per square inch.

Table No. 56 is similar to Table No. 55, but gives the unit shearing and compressive stresses at each joint if a vertical load equal to the total load of the earth above the sewer is assumed instead of the vertical load obtained from the Marston formula.

An inspection of the stresses, developed in the three ring arch, as listed in Table No. 55, as well as the equilibrium polygon, Figure 37, indicates that the three ring arch is safe provided voussoir 8 is considered as part of the abutment and additional masonry is placed back of this voussoir to resist the horizontal component of the arch thrust.



The authors have frequently specified that the first foot of the trench, measured upward from the horizontal diameter of the sewer (apparent spring line) should be backfilled with a lean mix of concrete. In unyielding soil, careful trimming of the trench will reduce the quantity of concrete required to a minimum, and this additional support can be added to the sewer at a relatively slight cost.

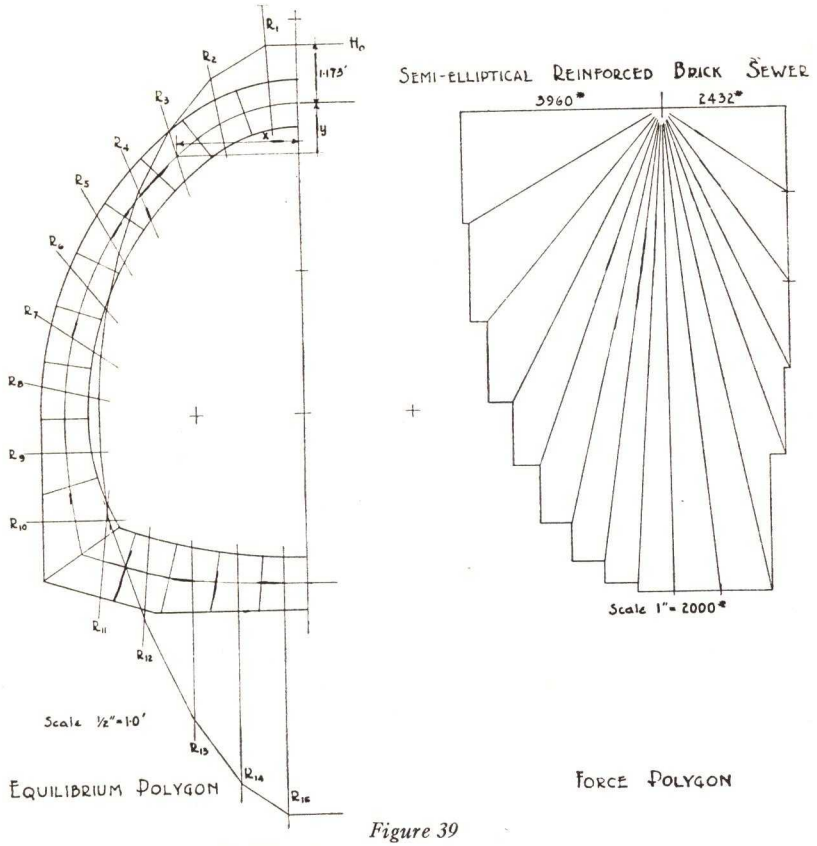



*Figure 39*

## 1416. GENERAL FORMULAS

**Formulas for Preliminary Design of Semi-Elliptical Sewer:**

As indicated in paragraph 1412, a sewer of semi-elliptical section is highly efficient structurally, since its form coincides closely with the line of resistance of the arch. If the normal sewage flow is in the neighborhood of one-third the maximum flow, this section is also efficient hydraulically and may frequently be used to advantage. A section of this type, designed by the engineers of the Sanitary District of Chicago, is shown in Figure 41 and its hydraulic properties are given in Figure 36.

The following formulas have been developed to aid in the preliminary structural design of sewers conforming to this section. They are applicable to reinforced brick masonry and to reinforced concrete, and the results will be found to check closely with those obtained from an analysis of the final design. The sewer is assumed to be symmetrically loaded, and the half section is considered as a curved beam acted upon by the external loads and by thrust and moment at the crown and at the center of the invert. Since the loading is symmetrical, the shears at these points are zero. The vertical and

horizontal components of the earth pressure are assumed to be distributed, in the manner shown in Figure 39, over the horizontal and vertical projections of the center line of the sewer ring. This is a slight approximation, since in the final analysis they are assumed to be distributed over the horizontal and vertical projections of the extrados. The approximate assumption is also made that the moment of inertia of the sewer wall is constant; this is exactly true only if the wall thickness is uniform throughout.

Taking the point E as origin, integral expressions can now be written giving the relative horizontal deflection, and the relative change in slope of the tangent, at the crown and the center of the invert. Both of these expressions must equal zero because of the symmetry of structure and loading. They are:

$$\int_E^A \frac{M'ds}{EI} = 0, \text{ and } \int_E^A \frac{M'yds}{EI} = 0$$

in which M′ is the bending moment at any point.

Assuming the moment of inertia as constant, EI may be dropped from both equations. Now, using the nomenclature of Figure 41, M′ may be expressed in terms of $M_i$, T, H, w, p, p′ and x and y, a separate expression being written for each of the four segments ED, DC, CB and BA. For example, in the segment, ED,

$$M' = M_i + Ty - \frac{wx^2}{2} - \frac{(p + p')\,y^2}{2} + \left(\frac{p'}{H}\right)\left(\frac{y^3}{6}\right)$$

Moments which produce compression in the extrados of the sewer are considered as positive.

These expressions must now be transformed to polar coordinates.

Again referring for example to the segment

$$\text{ED, } X = \frac{5H}{4}\sin\theta,\ y = \frac{5H}{4}(1-\cos\theta), \text{ and the limits of } \theta \text{ are O and } \beta$$

Substituting the transformed values for M′ in the integral equations and remembering that ds equals $pd\theta$, we obtain expressions involving the one variable $\theta$, which may be integrated and solved simultaneously for $M_i$ and T, the moment and thrust at the center of the invert:

$$M_i = 0.0741wH^2 - 0.0495pH^2 - 0.0293p'H^2 \quad \ldots\ldots (21)$$
$$T = -0.0222wH + 0.477pH + 0.332p'H \quad \ldots\ldots (22)$$

Here, H is in feet; p, p′ and w are in pounds per square foot; and $M_i$ and T are in pound-feet and pounds, respectively, per linear foot of sewer.

Moments at points A, B, C and D on the arch may be obtained from the general moment equation, $M = M_i + Ty + m$; where, m is the moment due to external forces between the origin and the point about which moments are taken and is opposite in sign to $M_i$:

$$M_D = -0.0194wH^2 - 0.0162pH^2 - 0.0069p'H^2 \quad \ldots\ldots (23)$$
$$M_C = -0.0583wH^2 + 0.0539pH^2 + 0.0320p'H^2 \quad \ldots\ldots (24)$$
$$M_B = +0.0334wH^2 - 0.0396pH^2 - 0.0199p'H^2 \quad \ldots\ldots (25)$$
$$M_A = +0.0519wH^2 - 0.0725pH^2 - 0.0306p'H^2 \quad \ldots\ldots (26)$$

$H_o$ may be determined from the formula:

$$H_o + T = \frac{2p + p'}{2}H \quad \ldots\ldots (27)$$

As an example of the application of these formulas, let it be required to design the sewer ring for the section shown in Figure 40. Presume that the depth from the surface of the ground to the crown of the sewer is 24.4 ft., and the weight of the backfill is 100 lbs. per cu. ft. The horizontal pressure will be taken as one-third the vertical.

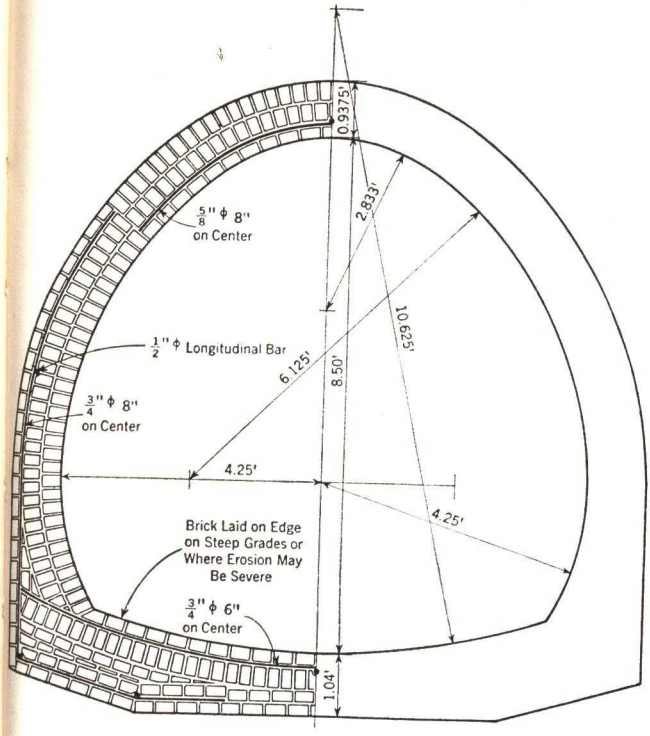
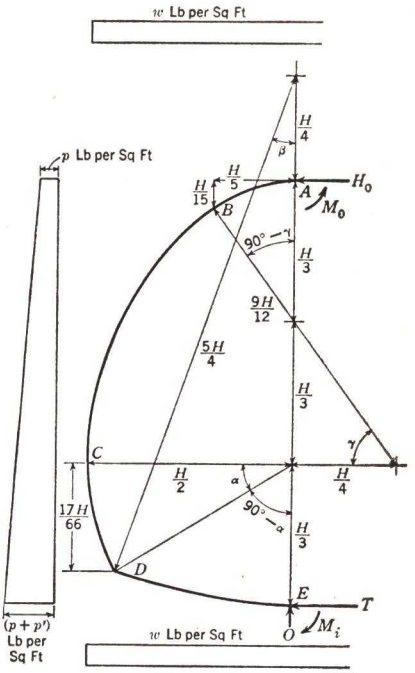



*Figure 40*




SEMI-ELLIPTICAL SEWER SECTION

*Figure 41*

Marston's formula for vertical pressure is $W = cw_1B^2$, in which W is the total load per linear foot of sewer; $w_1$, the weight of backfill in pounds per cubic feet; B, the width of the trench a little below the top of the pipe; and c, a coefficient depending upon the ratio of depth to width of trench and the character and condition of the backfill. In this example, B may be taken as 10.8 ft. and c as 1.70, giving a value of 19,850 lbs. for W.

$$w = 1{,}840 \text{ lbs. per sq. ft. and } p = \frac{W}{3} = 610 \text{ lbs. per sq. ft.}$$

On these assumptions the numerical values for moment and thrust at the crown are computed to be 4,630 lb.-ft. and 4,006 lb., respectively. A similar investigation for moment should be made at points, B, C, D and E.

The preliminary design of the ring, as shown in Figure 40 follows from a consideration of these values. A more precise analysis of this structure has been made, and the moments and thrusts thus determined are in close agreement with those obtained from the general formulas.

The maximum stresses (in pounds per sq. inch) developed in the section, due to combined compression and bending are:

| | Masonry | Steel |
|---|---|---|
| Crown | 488 | 11,700 |
| Haunch | 490 | 6,850 |
| Invert | 755 | 17,150 |

If the steel in the crown and haunch is reduced and the sewer is reinforced with ⅝ in. bars on 12-in. centers, the maximum stresses are:

| | Masonry | Steel |
|---|---|---|
| Crown | 550 | 16,700 |
| Haunch | 675 | 14,600 |

## 1417. DESIGN OF REINFORCED BRICK SEWERS

**(a) Assumptions:**

The following data will be assumed in this design:

1. Sewers to be constructed in yielding soil.
2. Depth from surface of ground to top of sewer (D) is 24.4 ft.
3. Weight of back-filling material 100 lbs./cu. ft. (partly compacted damp yellow clay).
4. Horizontal earth pressure equals ⅓ of vertical pressure.
5. Vertical pressure equals that obtained from Marston's formula, $W = cwB^2$.
6. Semi-elliptical section designed by engineers of Sanitary District of Chicago. H (inside dimension) equals 8 ft. 6 in.

Since this sewer is to be constructed in yielding soil, it will be assumed that the sewer will be reinforced. In such locations, plain masonry arches must be supported by relatively massive abutments and reinforced masonry structures which can be designed as a whole or a unit are usually more economical.

The following values should be substituted in the Marston formula to obtain the vertical earth pressure, w:

$B = 10.8$ ft.
$w = 100$ pounds per cubic ft.
$D = 24.4$
$D/B = 2.26$
$c = 1.70$ (Table 48)
$W = cwB^2 = 19850$ pounds per linear ft.

The vertical earth pressure per square foot (w in above formula) equals 1840 pounds. Horizontal pressure (p) equals ⅓ the vertical pressure, equals 610 pounds per square foot.

Substituting values for w, H, and p in equations 21 to 27 incl., the following thrusts and moments are obtained:

$T = 2386$ lbs.
$H_o = 4006$ lbs.
$M_i = 9602$ ft. lbs.
$M_A = 4630$ ft. lbs.
$M_D = -4112$ ft. lbs.
$M_C = -6705$ ft. lbs.
$M_B = 3366$ ft. lbs.

The working stresses and calculated data used in the preliminary design are:

$f_s = 16{,}000$ lbs. per sq. in.
$f_b = 700$ lbs. per sq. in.
$n = 15$

The following are correct when bending only is assumed:

$p = .0087$
$k = .3962$
$j = .8679$
$K = \frac{1}{2} f_b kj = 120.4$

The actual stress in the sewer wall is a combined stress due to both compression and bending. However, for the preliminary design only the bending stress is considered, and the thickness of masonry and steel required is determined approximately from the following formulas:

$$f_s \text{ (stress steel)} = \frac{M}{A_s jd}$$

$$f_b \text{ (stress masonry)} = \frac{2M}{jkbd^2}$$

Substituting in these formulas, we obtain the following values of d, effective thickness of the sewer ring, at the various points for which moments have been determined:

Crown 6.2 inches
Point D 5.8 inches
Point C 7.5 inches
Center of invert 9.0 inches

The preliminary design, illustrated on Figure 40, is based upon these values.

The method used in the analysis of this design is the method for indeterminate structures as prepared for Metcalf and Eddy by Professor Arthur W. French and described in "American Sewerage Practice", Volume I, page 105.

This method of analysis is somewhat more complicated than the methods which make use of the line of thrust theories and will not be described in detail. For a complete description, the reader is referred to the reference given above, or to Turneaure and Maurer "Principles of Reinforced Concrete Construction" in which a similar theory, known as the elastic theory, is discussed.

In this method of analysis, the moment of inertia of the sewer wall is assumed to vary directly with the thickness cubed and the moment of inertia of the steel is neglected. This is an approximation since the true moment of inertia is the moment of inertia of the transformed section equal to the moment of inertia of the masonry plus "n" times the moment of inertia of the steel.

However, in view of the fact that the amount of the external forces acting upon the sewer can only be approximated, too great refinement in the methods of design is not justified.

From this analysis, $H_o$ and $M_A$ are determined as:

$H_o = 3960$ lbs.
$M_A = 4648$ ft. lbs.

The eccentricity of the crown thrust is obtained by dividing $M_A$ (moment at the crown) by $H_o$ which gives the value 1.173 ft. Since the moment is plus, the eccentricity is also plus and is measured above the center of the crown. Having the crown thrust and the eccentricity, the force and equilibrium polygons may be constructed for the arch, as indicated in Figure 39.

Moments at each joint of the arch may be obtained from the equation, $M = M_o + H_o y + m$. These values are listed in column 2 of Table 57. The thrusts for the various joints, scaled from the force polygon, are listed in column 3. Column 4 gives the eccentricities at each voussoir obtained by dividing the moment by the thrust and

checked by sealing the equilibrium polygon. The shears for each voussoir scaled from the force polygon, are listed in column 5.

Since the steel required to resist tension is usually made continuous throughout an entire section of an arch, it will be noted from Table 57 that the bending moment at voussoir 1 of 4752 ft. lbs. will determine the quantity of steel required for this section. The bending moment at voussoir 9 minus 6323 lbs. determines the quantity of steel required for this section and the bending moment at the invert, 11648 lbs., fixes the amount of steel in the invert.

The following equations for determining the stresses in the masonry and the steel due to combined compression and bending are presented by Turneaure and Maurer in "Principles of Reinforced Concrete Construction", Third Edition, page 104:

$$f_s = nf_c\left(\frac{d}{kh} - 1\right)$$

$$\tfrac{1}{2}f_c\, bkh\left(\frac{h}{2} - \frac{kh}{3}\right) + f_sA\left(d - \frac{h}{2}\right) = M$$

$$\tfrac{1}{2}f_c\, bkh - f_sA = N$$

## TABLE NO. 57

## BENDING MOMENTS, THRUSTS, AND SHEARS — SEMI-ELLIPTICAL REINFORCED BRICK SEWER

| Col. No. 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | Steel reduced crown & haunch 9 | 10 |
|---|---|---|---|---|---|---|---|---|---|
| Joint | Moment ft. lbs. | Thrust lbs. | Eccentricity ft. | Shear lbs. | k | $f_b$ #/sq. in. | $f_s$ #/sq. in. | $f_b$ #/sq. in. | $f_s$ #/sq. in. |
| Crown | 4596 | 3975 | 1.16 | 00 | | | | | |
| 1 | 4752 | 4150 | 1.14 | 1540 | .29 | 488 | 11,700 | 550 | 16,700 |
| 2 | 3788 | 5020 | .75 | 2050 | | | | | |
| 3 | 2402 | 6090 | .39 | 2350 | | | | | |
| 4 | 483 | 7010 | .07 | 2530 | | | | | |
| 5 | −1678 | 8000 | −.21 | 2550 | | | | | |
| 6 | −3723 | 8790 | −.42 | 2150 | | | | | |
| 7 | −5376 | 9290 | −.58 | 1520 | | | | | |
| 8 | −6137 | 9520 | −.64 | 800 | | | | | |
| 9 | −6323 | 9610 | −.66 | 400 | .40 | 490 | 6850 | 675 | 14,600 |
| 10 | −5921 | 9580 | −.62 | 2020 | | | | | |
| 11 | −2225 | 4600 | −.48 | 5640 | | | | | |
| 12 | 3118 | 3760 | .83 | 4310 | | | | | |
| 13 | 7241 | 3080 | 2.35 | 2920 | | | | | |
| 14 | 10156 | 2626 | 3.87 | 1400 | | | | | |
| 15 | 11648 | 2400 | 4.85 | 100 | | | | | |
| Invert | 11648 | 2420 | 4.82 | 00 | .31 | 755 | 17,150 | | |

In the above equations, h equals the total thickness of the sewer wall; N is the thrust; M the bending moment; and e, the eccentricity. The other symbols have the same meanings as when used in the formulas for reinforced concrete for simple flexure.

By solving the above equations simultaneously, the values of k, $f_c$, and $f_s$ may be determined. The solution of these equations is somewhat laborious as the final equation for k is a cubic equation which must be solved by trial. Values of k, $f_s$, and $f_c$ for the critical points in the sewer ring are listed in columns 6, 7 and 8 of Table 57. It will be noted that the stress developed in the steel in the arch of the sewer is considerably below the allowable working stress and some reduction in the quantity of the steel may, therefore, be warranted. However, due to the combined stress, the allowable working stress of the masonry will usually be the controlling factor and stresses developed in the steel will be relatively low.

The stresses, as indicated in Table 57, which will be developed in the invert are high in both the steel and the masonry. However, the distribution of the external loads assumed is probably more unfavorable than would ordinarily be found and stresses based upon these assumptions are, therefore, greater than would actually be developed.

Shearing stresses in sewers are usually well below the allowable working stress for the masonry. Unit shears for this section may be determined by dividing the total shear at each joint by the area of the joint.

Experience has indicated that longitudinal reinforcement to resist temperature stresses is not required in a brick sewer. However, in order to assist in the fabricating of the transverse reinforcement and as an added factor of safety, the ½ inch longitudinal bars, indicated on Figure 40, have been included.

Figure 40 is a working drawing of a semi-elliptical sewer. The steel shown in the semi-elliptical sewer is that assumed in the preliminary design, although as indicated above, a redesign would probably permit the use of less steel.

It is not the intention that the foregoing designs should in any sense be considered as standard designs or standard sections. They are presented only to illustrate the methods which should be followed in the design of brick sewers and while it is sometimes difficult to determine in advance the soil conditions which will be encountered during the construction of a sewer, it is believed that an analytical check of the stresses which will be developed in the sewer walls, based upon what appear to be reasonable assumptions as to soil conditions, will result in better and frequently more economical designs.

The method of analysis outlined in Section 1416 may be used to advantage in investigating a revised design. Assume that the steel in both the crown and haunch of the sewer, Figure 40, is reduced to ⅝" bars on 12" centers. For joint 1, the following data are available or may be computed:

Joint 1
Thrust at Joint 1: T = 4150 lbs.
Moment at Joint 1: $M_o$ = 4752 ft. lbs.
Eccentricity from center of section: e = 1.14 ft.
Total depth at Joint 1: h = 11.25 inches = .9375 ft.
Effective depth (to steel): d = 8.50 in. = .7075 ft.

$$\text{Moment arm about steel: } e_1 = 1.14 + \frac{(5.63-2.75)}{12} = 1.38 \text{ ft.}$$

Moment about steel: $M_s$ = 1.38 x 4150 = 5720 ft. lbs.

If stresses in masonry and steel are assumed as 650 and 18,000 lbs. per sq. in., respectively, we obtain from tables:

K = 100.8 (unit-lbs. per sq. in.)
p = .0063

The required effective depth, d, is obtained from the equation

$$bd^2 = \frac{M}{K} = \frac{5720 \times 12}{100.8} = 681, \text{ and since } b = 12 \text{ inches,}$$

$$d^2 = 681/12 = 56.7$$

$$d = 7.5 \text{ inches}$$

If this value of d is used, the moment about the steel will change and must, of course, be recalculated. If, however, d is made 8.5 inches, the value originally assumed, the stress in the masonry and the required steel are determined as follows:

$$K = \frac{M}{bd^2} = \frac{5720 \times 12}{8.5 \times 8.5 \times 12} = 79.2$$

If $f_s$ is assumed as 18,000 lbs. per sq. in., we obtain from tables:

$K = 79.2$
$f_b = 558$ lbs. per sq. in.
$f_s = 18{,}000$ lbs. per sq. in.
$p = .00492$

Required steel area may then be determined:

$$A_s = .00492 \times 8.5 \times 12 - \frac{4150}{18{,}000} = .502 - .231 = .271 \text{ sq. in. per ft.}$$

If the exact values of d and $A_s$ obtained from the above equations are used in the design, the stresses in the masonry and steel will be the assumed stresses, namely, 558 lbs. per sq. inch and 18,000 lbs. per sq. inch respectively. Frequently, however, standard sizes of masonry units and reinforcing bars will make it desirable to change the values of d and $A_s$ slightly from the theoretical values. Common practice is to adopt the nearest standard size above the theoretical. If this is done, both the stress in the masonry and the stress in the steel will be less than the assumed values.

If it is desired to determine the actual stresses in an assumed design, these stresses may be obtained by means of the equations listed in Section 1417, or by the above method through successive approximations. By the latter method, when the stresses assumed give values of d and $A_s$ equal to the actual values, then the assumed stresses are the actual stresses. Actual stresses for the above assumptions are given in columns 9 and 10 of Table 57.

# CHAPTER 15

# MISCELLANEOUS STRUCTURES OF REINFORCED BRICK MASONRY

## 1501. REINFORCED BRICK MASONRY BUILDINGS

What is believed to be the first industrial building in the United States to be constructed with 4-in. reinforced brick masonry walls was completed in June, 1932 by a large oil refinery in Illinois. The details of the method used to reinforce the walls and columns of this interesting building were described in Civil Engineering for April, 1933, by Mr. H. S. Haworth, Structural Engineer, Standard Oil Company, (Indiana).

As a result of his experience at the refinery with other buildings having walls of reinforced concrete, thick plain brick, and corrugated asbestos, he believes that reinforced brick masonry for such construction may be expected to come into general use because of its economy and good appearance. It is also remarkably strong, as shown by the fact that the columns of the compressor building carry a 12-ton crane. The cost data included will be useful for comparison with other types of construction.

The building is approximately 45 ft. wide, 78 ft. long and 20 ft. high. Mr. Haworth goes on to say that "the brickwork was started on May 4, 1932, and completed on June 3 of the same year. All the walls and columns of this building are of reinforced brick masonry.

"The footings under the columns in the main building are of reinforced concrete and those under the three columns in the outside wall of the switch room are of reinforced brick masonry. Dowels of the same size and number as the vertical bars in the columns project 2 ft. 6 in. above all the footings into the brick columns above.

"The reinforced brick columns were started from the concrete or brick footings and laid to a point approximately 6 in. below the finished exterior grade. The curtain walls were started from plank forms supported on dry brick piers set between the columns. It was necessary to use the plank forms because the original ground was about 2 ft. below the finished floor grade. The curtain walls were bonded to the columns.

"All the mortar joints have a minimum thickness of $\frac{5}{8}$ in. and are weather struck. The brick were laid from a plank staging supported on wooden trestles. All brick were first thoroughly wet with a hose and then were laid by the Western or "stringing mortar" method, that is, mortar was spread along the top of the brickwork for 3 or 4 ft. before the masons started to lay the next brick course. All the brickwork was laid with "shoved joints." Masons were not permitted to furrow the bed joints.

"The reinforcing steel was set by the masons, who wired the vertical bars in the columns to the dowels in the footings and to a wood template about 10 ft. above the ground. These bars were plumbed and then braced two ways with diagonal wooden braces. The column ties were put around the vertical bars before they were erected and the masons slid one tie down into a mortar joint when required. The horizontal wall bars were laid on the dry brick and were held in place by small pats of mortar until the next brick course was laid. All the brickwork was wet thoroughly twice a day for ten days after it was laid in order to facilitate the hydration of the mortar.

**Methods of Reinforcing**

"The side columns of the main building are 23 by 27 in. up to the crane girder seat, above which they are 23 by 14 in. The reinforcing consists of six ¾-in. round bars, three in the outside and three in the inside of each column, located about 4½ in. from each face, as shown in the illustrations. The three inside bars extend from the footing to 3 in. below the crane girder seat.

"Three ¾-in. round dowels were set 4½ in. from the inside face of the 14-in. section of each column so as to project 2 ft. 6 in. above and below the crane girder seat. The bars in the column above this point were wired to these dowels. The eight horizontal mortar joints immediately below the crane girder seats were reinforced by placing rectangular pieces of welded wire mesh in the mortar as the brick were laid. The dimensions of the mesh were about 3 in. less than the dimensions of the column, and the wires were No. 8 gage, spaced 4 in. from center to center both ways. Throughout the remainder of the column ¼-in. round ties were used, spaced about 9½ in. from center to center.

"At the crane girder seat, a ¾-in. steel plate was cut to fit around the three dowel bars and set to grade in mortar. The crane girder was electrically welded to this plate. A similar procedure was followed for the bearing plates of the roof trusses. About 6 in. of the outside ends of the roof trusses were bricked into these columns.

"The columns at the sides of doors and along the outside of the switch room are 14 by 23 in. and were reinforced with four ¾-in. round vertical bars and ¼-in. round ties spaced about 9½ in. from center to center.

"The walls in the two bays between the switch room and the main building contain three tiers of brick, and are approximately 14 in. thick. The horizontal mortar joint above the first course of brick is reinforced with six ⅜-in. round bars, the outside bars being about ½ in. from each face of the wall. Above this point, two ⅜-in. round bars were used in each second horizontal joint, at a vertical spacing of about 6⅜ in. from center to center. These bars were set in the mortar joint about ½ in. from each face of the wall, which was made quite heavy because it was anticipated that the switching equipment would cause considerable vibration.

"All other walls contain only one tier of brick, and are nominally 4 in. thick. The horizontal mortar joint above the first course of brick is reinforced with two ⅜-in. round bars, placed about ½ in. from each face of the wall. Above this the reinforcing is in every third horizontal mortar joint, at a vertical spacing of about 9⅝ in. from center to center. To provide for positive moment due to wind loads, one ⅜-in. round bar was placed about ½ in. from the inside face of the wall and extended between the brick columns. To provide for negative moment at the columns, bars with a length of one-half the span were set so that their centers would come at the center of the columns. This gave negative reinforcement for one-quarter of the span. These bars were located about ½ in. from the outside face of the wall. At the corners of the building, one ⅜-in. round bar was set ½ in. from the outside face of the wall and extended 4 ft. along each wall. In the first joint above the head of windows and doors, two ⅜-in. round bars were set, in order to provide reinforced brick masonry lintels.

"The frames for the doors and windows were made from 3 by 3 by ¼-in. steel angles, and the side angles were punched with slotted holes 19 in. from center to center, so as to coincide with every sixth mortar joint in the brickwork. Carriage bolts, ⅜ by 6 in., were put through these holes and bonded into the mortar joint as the brickwork progressed. Stock steel factory sashes and mullions were welded to the window frames.

"The cost of this building, exclusive of the foundations for the compressors, the 12-ton traveling crane, the heating equipment, and the electric wiring, was 9.2 cents per cu. ft. or $1.875 per sq. ft. The compressor house was only a small part of the construction that was being carried on in that part of the refinery. It was necessary to put up

this building in parts so that temporary railroad tracks and other construction equipment could be used. A comparison of the costs of various types of curtain walls used at this refinery is given in Table 58.

TABLE 58

COSTS OF VARIOUS TYPES OF CURTAIN WALLS

| Type | Cost per Sq. Ft. |
|---|---|
| 4-in. reinforced brick, using No. 2 pavers (columns not included) | 23.0 cents |
| Corrugated asbestos and steel girts (columns not included) | 27.0 cents |
| 8-in. plain common brick | 47.5 cents |
| 6-in. reinforced concrete, using metal panel forms | 56.3 cents |

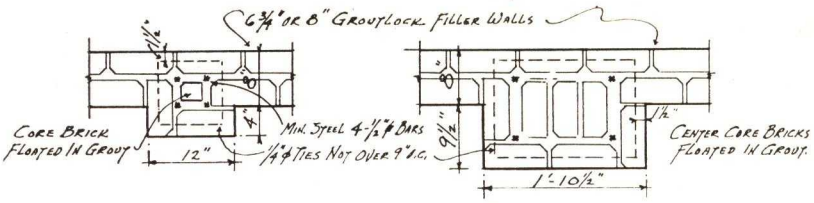
"On the morning of July 26, 1932, rain to the amount of 1.49 in. fell between 4:00 and 7:00 a.m. The west wall of this building was wet on the inside from a point about 12 ft. above the floor down to the floor. No other parts of the building seemed to be affected in this way. Investigation revealed that water was running off the roof, which has no gutter at the eave, and falling on to some large pipes, the tops of which are about 13 ft. above the floor. The water splashed from the pipes on to the 4-in. brick walls and apparently seeped through the mortar joints. Several methods of remedying this situation were suggested, among them: (1) to provide a gutter along the eaves; (2) to waterproof the exterior of the brickwork with a colorless waterproofing compound; and (3) on future work, to use a tooled concave joint in the brickwork.

"The leakage has now practically stopped, with no changes or additions to the original construction. Apparently, the water seeping through the joints has tended to "heal" the cracks through which the water was leaking. On other buildings, the third of which was completed in February 1933, concave tooled joints were used. There has been no evidence of leakage on any of these buildings. However, none of them has been subjected to as severe conditions as the compressor house.

"It appears to me, after my experience with this building, that the use of reinforced brick masonry can be expected to become quite general. This is especially true of industrial work because it is economical to construct; it presents a more pleasing appearance than corrugated iron or corrugated asbestos; the heat loss through a 4-in. brick wall is about 60 per cent of that through corrugated asbestos; and its maintenance will probably be small. It also appears that reinforced brick masonry may be expected to supplant reinforced concrete in many places where there are large vertical surfaces, because it obviates the necessity of providing forms for such surfaces."

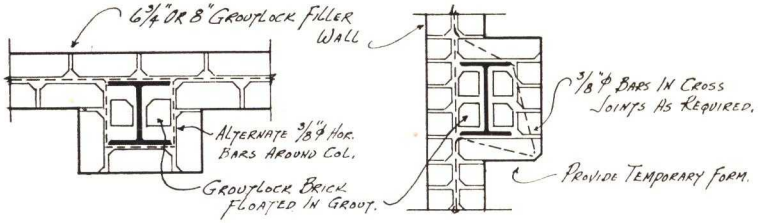
There have been a number of reinforced brick masonry buildings constructed in recent years. A large percentage of such buildings are in California. Most construction on the Coast is required to be earthquake-resistant and practically all brick masonry is designed and reinforced accordingly.

The engineers of the Simons Brick Company were probably the first to promote the use of reinforced, grouted brick masonry. As mentioned in Chapter 9, a large number of buildings, so constructed, have gone through one or more tests of a severe nature with a perfect record. At first this construction was used only in dwellings and the smaller store and office buildings. As more designers came to realize the many advantages of reinforced brick masonry, we find it used more generally in all types of buildings. Some of the outstanding examples are several large school buildings, one of which is the Vermont Avenue School in Los Angeles, described in Section 927. Another more recent one of reinforced brick masonry on a large scale is the Walt Disney Studio development.



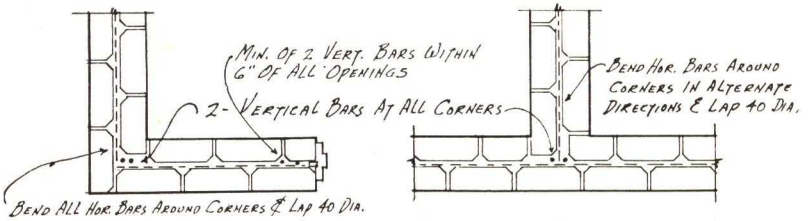

· PLANS OF GROUTLOCK PIERS OR COLUMNS ·

MAY BE MADE PRACTICALLY ANY SIZE - ILLUSTRATIONS SHOW ONE SMALL & EXTREMELY LARGE.
FOR VERT. LOADS IN L.A. FIGURE 175# x OVER ALL AREA + 29 x AREA OF VERT. STEEL x 175# TOTAL VERT. LOAD.

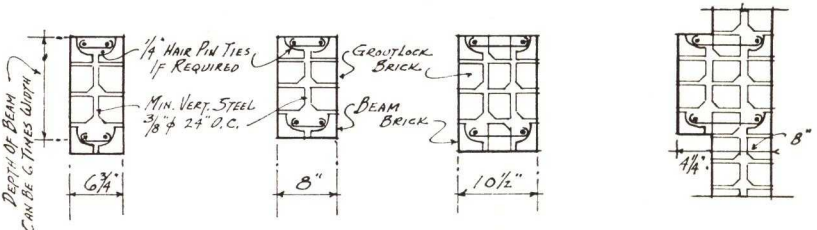



· COLUMN · · BEAM ·

STRUCTURAL STEEL COLUMN & BEAM WITH GROUTLOCK FILLER WALL ·




PLAN - TYPICAL CORNER & JAMB. PLAN OF WALL INTERSECTION



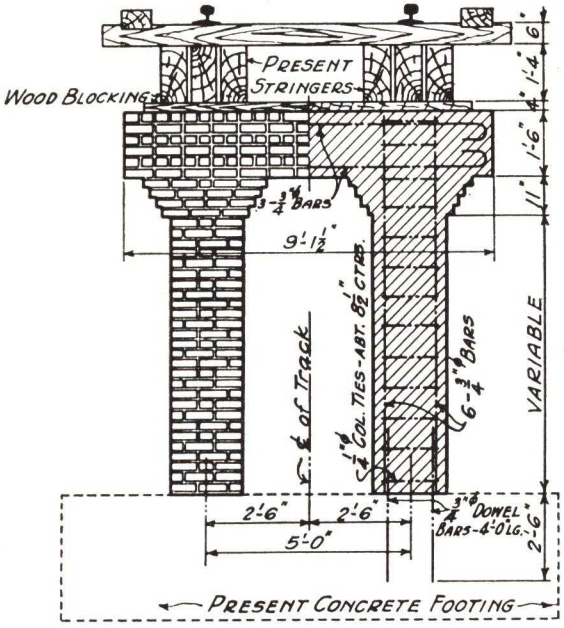

TYPICAL GROUTLOCK BEAM & LINTEL CONSTRUCTION

*Figure 42*

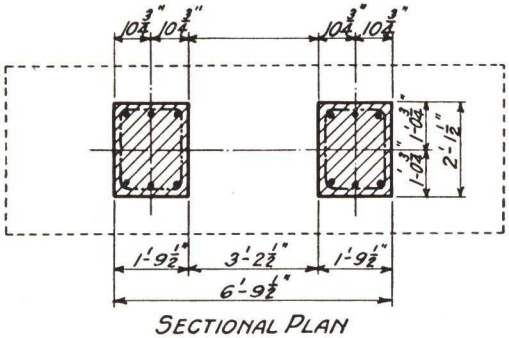
*Typical details of Groutlock reinforced brick masonry*

## 1502. R-B-M PIERS FOR A RAILROAD TRESTLE

Reinforced brick masonry piers, used for a railway trestle at Wedron, Illinois, were designed by Mr. Hugo Filippi, Civil Engineer, and reported in Building Economy, March-April 1932. The construction replaced an industrial trestle 293 feet long which the company had built in 1922 of the usual type of framed timbers. The old framed supports of the trestle ranged from 7 to 11 feet in height and rested upon concrete footings. These footings were utilized as a support for the new brick piers. Deep holes were drilled into these footings and into the holes steel dowel bars were securely grouted into place. This detail, as well as the reinforcing details and general design of the piers, is shown in an accompanying illustration.




HALF ELEVATION ~ HALF SECTION




SECTIONAL PLAN

DETAIL OF REINFORCED BRICK MASONRY PIERS

Acting upon instructions from the bridge department of the railroad, an individual car loading, totaling 210,000 pounds, was used in designing the piers. The minimum pier thickness is substantially two feet, which corresponds with the railroad company's standard minimum requirements for bridge or trestle piers. This thickness, however, is much greater than would have been required on the basis of safe load-carrying capacity.

The track ties and stringers were found to be in satisfactory physical condition. To carry the track load while a brick pier was being built, a wide cribbing of second-hand track ties was built tightly under the existing stringers of the spans adjacent to the pier. This done, the old wood support was removed; the holes for the dowel bars drilled; the dowels solidly grouted into place and brickwork on the piers commenced immediately.

Progress in building the piers was unusually rapid. Two bricklayers built, on an average, three piers every two days, and the track load was transferred to the new piers within from 24 to 48 hours after their completion. The number of brick laid per 8-hour day per bricklayer averaged 932. The average number of bricklayer hours, common labor hours, and the quantity of mortar materials required to lay up 1,000 brick are shown in the statement following:

| | |
|---|---|
| Bricklayer hours (including placing of reinforcing rods) | 8.6 |
| Sacks of portland cement | 5.86 |
| Common labor hours | 12.9 |
| Sacks of lime | 5.21 |
| Tons of sand | 1.21 |

The mortar actually used was essentially a 1-1-6 cement lime mixture. The average number of brick laid in each pier was 1,243, and the total number of brick used in the job was 26,100.

Only 14 working days were required to complete the entire job, ready for use, including the time necessary for drilling holes, removing existing wood supports, doing the necessary cribbing and shoring of the stringers and track, and for building the piers complete, including the placing of reinforcement. Both a typical pier and a general view of the completed trestle are shown in the illustrations.

## 1503. R-B-M STORAGE BINS AND TANKS

Structures for the economic storage and handling of materials play an important part in modern plant design. From the mine or field to the finished product, the problem of storage is important, particularly for such raw materials as coal, ore, grain, sand, gravel and clay. At the plant of the Wedron Silica Company, Wedron, Illinois, in June, 1931, storage bins were constructed entirely of reinforced brickwork. These storage bins were designed by Mr. Hugo Filippi, and reported in Building Economy, March-April 1932.

The ground space, fixed by economical operation and by existing plant conditions for the location of the bins, was small in area and irregular in shape. It consisted of a trapezoidal plot of ground only 12 feet wide at one end, 35 feet wide at the other end and only 50 feet in length, comprising in all an area of only 1,175 square feet. The basic problem was to provide maximum storage capacity for three grades of sand at minimum cost. This was accomplished by providing two circular bins — one 25 ft. 1-inch inside diameter, divided into two compartments by a center wall, and the other bin 16 feet $2\frac{7}{16}$ inches inside diameter. Both bins are 52 feet high and provide a combined storage space for 2,100 tons of sand. On top of the bins, and developing the full area of the projected ground space, a structural steel frame sand screening house was built. In this house, which consists of two floor levels, are located the sand screens, conveying machinery, chuting devices, secondary storage bins and other appurtenances.

The new sand bins are built entirely of reinforced brick masonry. All walls rest on stepped brick footings which in turn rest on a bed of mixed sand and gravel about 5½ feet below grade. The circular wall of the 25-foot bin is 13½ inches for a height of 22 feet above the top of the foundation and the balance of the wall, 30 feet, is 9¼ inches thick. The center dividing wall in the 25-foot bin is 21 inches thick for the first 14 feet above top of foundation; 17 inches thick for the next 22 feet, and 13 inches thick for the upper 16 feet. As shown in Figures 1 and 2, the ends of this center wall were splayed or flared. The circular walls of the 16-foot bin are 9¼ inches thick from bottom to top.

The principal reinforcement consists of bent horizontal rings in the circular walls and of straight and bent horizontal bars in the center wall. In addition, however, the lower 14 feet of the center wall was reinforced vertically. Reinforcing bars in reinforced brickwork can be spaced easily and accurately, but what is more important, the intended size and location of each bar may be checked visually in the field as the work progresses when and as each bar is being placed. Since the disposition of reinforcement in reinforced brickwork design is a function of the thickness, width or length of a brick, it follows that danger of bar displacement is generally absent. If reasonable care is taken in laying out the bonding of the brick courses, cutting, clipping, or splitting of bricks can easily be eliminated, thereby effecting decided economy in cost of construction. It may be observed that no cutting of brick was required and that the mortar joints were laid out so as to provide proper coverage for all reinforcing steel.

To support the first run of vertical reinforcement, templates of wood were temporarily erected. To these templates, the vertical rods were attached by means of nails and held in position until well bricked in. Subsequent runs of vertical bars were, at the proper time, spliced to bars already in place and lightly wired to lengths of bent reinforcing bar rings temporarily placed at a convenient height above the working platform. Thus the correct spacing for the newly placed vertical steel was maintained and swaying was prevented. All horizontal steel was placed directly on or in freshly laid beds or channels of mortar.

The construction scaffold used was extremely simple. Instead of the customary braced tower, a loose plank platform, cut to fit the inner diameter of the bins, was employed. This planking was supported by a suitable number of 4x6-inch timbers (commonly called "put-logs") let into the brickwork. This platform arrangement is very economical and convenient and follows the practice commonly used in constructing brick chimneys. These platforms carried the full load of workmen, material stock and the tripod arrangement used for hoisting.

The reinforced brickwork was built up from these platforms to a height of about 5½ feet. When this limit of working height was reached, a second set of "put-logs" was placed on the fresh brickwork, the loose planking was transferred to the new working level, and bricklaying was resumed at once.

To overcome the difficulties imposed by limited working space, and other local conditions surrounding the job, three construction openings were provided and all materials were hoisted from the inside of the bins. When the bins were completed, the reinforcing steel projecting into the openings was spliced out and the openings were neatly bricked up.

All mortar used in this job consisted of one part of portland cement and three parts of screened torpedo sand, to which was added 18 per cent of high-magnesium, ground lime putty. This mortar mixture, obtained by successive experiments provided suitable plasticity and had all desired physical properties.

A very careful classified daily check was kept on all labor employed, the amount of work performed and materials used. The figures given below are taken from and based on actual job records.

## DATA ON LABOR PERFORMANCE AND MATERIALS REQUIRED TO BUILD SAND STORAGE BINS

| Part of Structure | Total Number of Bricks Laid | Bricklayer Hours per Thousand | Common Labor Hours per Thousand | Number of Bricks Laid per 8-Hour Day Per Man |
|---|---|---|---|---|
| Foundation | 32,100 | 3.67 | 4.24 | 2,175 |
| 13½-inch Wall — 25-foot Bin | 33,200 | 6.40 | 5.35 | 1,225 |
| 9¼-inch Wall — 25-foot Bin | 29,700 | 7.12 | 7.84 | 1,110 |
| 9¼-inch Wall — 16-foot Bin | 35,800 | 8.12 | 8.82 | 985 |
| 21-inch Center Wall — 25-foot Bin | 9,700 | 8.71 | 6.28 | 920 |
| 17-inch Wall — 25-foot Bin | 11,700 | 6.72 | 7.90 | 1,190 |
| 13-inch Wall — 25-foot Bin | 6,700 | 7.03 | 8.45 | 1,135 |

Moving Scaffold — 1.26 hours common labor per M bricks laid.
Bending and carrying steel — 2.71 hours common labor per M bricks laid.
Cement required per M bricks laid — 8.48 sacks.
Lime required per M bricks laid — 1.52 sacks.
Sand required per M bricks laid — 1.62 tons.
Reinforcing Steel required per M bricks laid — 25.9 pounds.*

*For walls above grade only.

### 1504. REINFORCED BRICK MASONRY SILOS

#### (a) Foundations

The depth of the silo foundations is governed by the special circumstances of each case. In northern climates, the bottom of the footing should be below the frost line so that wet earth beneath cannot freeze and by its expansion heave up the wall. The frost line varies in different parts of the country, the further north, the lower the frost line. In Ohio it is about 30 inches below grade. In northern Wisconsin about 36 inches, and further north, it ranges from 4 feet to 4 feet 6 inches. The frost line being determined (or disregarded in warmer states) the firmness of the earth at the level selected is the next consideration. In some cases, where the sub-soil is not firm, it is worthwhile to go down an extra foot or so to reach firm ground. Too great a depth is not desirable, however, as this increases the labor of getting the silage out near the bottom and in unstable soil the footings may be made wider instead of carrying them down to an excessive depth. For soils of ordinary firmness, 16-inch width is ample for the footings. The excavation for the walls of the chute should be carried down to the same level as for the silo. The footings of the chute need not be as wide as the footing for the silo — an 8-inch width is generally sufficient. The floor of the chute should be on the same level as the floor of the feeding room.

#### (b) Drainage

When a silo is built without a roof, it is essential to provide for draining away the excess water that accumulates at the bottom, due to the top being open to the rain and snow. With a roofed silo, if the silage material is too dry when the silo is filled, it is necessary to add water to it, any excess of which should be free to drain away. When the silage material has been cut too soon and consequently is too green and contains too much moisture, it is again necessary to provide drainage. In the case of either a roofed or unroofed silo a drain should be provided in which should be placed a tight trap (generally a bell trap is used) to prevent air entering the silo through the drain. There are cases on record where several tons of silage at the bottom of silos have spoiled because of air entering through the drains.

If the silo is constructed in clay or water-bearing soils, it is desirable also to install a footing drain around the outside of the silo. This drain will tend to reduce seepage into the silo pit and will increase the stability of the soil upon which the footings rest.

**(c) Floor**

Where silos are built on dry ground and the foundations are deep enough to prevent rats from burrowing underneath, the earth floor may give satisfactory service if covered with a thin layer of straw. However, a masonry floor is desirable since it is vermin proof and permits the silo to be thoroughly cleaned.

A very economical floor may be constructed of brick laid flat. The method of constructing such a floor is outlined in Part V, "Construction".

**(d) Design of Walls**

Silo walls should be designed to resist the bursting pressure caused by the silage. Investigations made some years ago by Professor F. H. King of the University of Wisconsin indicate that this pressure is equal to 11 pounds per square foot for each foot of height of silage. This is considerably less than hydrostatic pressure or the pressure due to a column of water which is 62.4 pounds per square foot for each foot of height.

The tension or stress in the side walls of a silo or any circular container due to bursting pressure is obtained from the formula,

$$S = \frac{pDH}{2}$$

where "S" is the tensile stress in the side walls,"D" the diameter of the silo in feet, "H" the height in feet of the section of the side wall under consideration, and "p" the pressure at the center of the height (H) in pounds per square foot.

For a silo, "p" is equal to 11h where "h" is the height in feet from the center of the section to the top of the silage.

If the height (H) of the section being analyzed is one foot, the formula to be used in the design of a silo becomes:

$$S = 11hD/2 = 5.5hD$$

As an example, assume it is required to determine the reinforcement in a 1 ft. section of the wall, the center of which is 44 ft. below the top of the silage. Assume that the diameter of the silo is 12 ft. Substituting in the above formula:

$$S = 5.5hD$$
$$= 5.5 \times 44 \times 12 = 2904 \text{ lbs.}$$

Therefore, the area of the steel included in this 1 ft. section must be sufficient to resist a pull or tension of 2904 pounds. It would be desirable to distribute the steel uniformly throughout this 1 ft. section. However, it is perfectly satisfactory to include it in one or more of the mortar joints. It is recommended, however, that for ordinary reinforcement the spacing should not exceed one foot due to the bursting pressure which acts upon the brick masonry between the layers of reinforcement.

In designing a reinforced masonry structure, sufficient steel is provided to carry the entire tensile stress. The cross-sectional area of the steel required will depend upon the stress in pounds per square inch that the steel is designed to carry. The recommended working stresses for reinforcing steel vary from 16,000 to 20,000 pounds per square inch, and, while steel is available which will resist considerably greater stresses than the above, the use of high unit stresses in the reinforcement for a silo or water tank is questionable practice due to the elongation of the steel under stress and the consequent cracking of the masonry.

Steel reinforcement in a silo 12 feet in diameter, if subjected to a unit stress of 30,000 pounds per square inch, will elongate approximately ½ inch. Since the masonry

## TABLE NO. 59

# TABLE OF STEEL REINFORCEMENT FOR REINFORCED BRICK MASONRY SILOS

**Indicating Number of No. 9 Wires, No. 3 Wires, or ⅜″ Round Rods per Joint**

**One ⅜″ Round Rod may be Considered Equal to Two No. 3 Wires or Equal to Six No. 9 Wires**

| Distance From Top of Silo | Brick Joint | Diameter Inside of Silo 10′ | 12′ | 14′ | 16′ | 18′ |
|---|---|---|---|---|---|---|
| 0 | 288 | 1 | 1 | 1 | 1 | 1 |
| | 287 | 0 | 0 | 0 | 0 | 0 |
| | 286 | 0 | 0 | 0 | 0 | 0 |
| | 285 | 0 | 0 | 0 | 0 | 0 |
| 11″ | 284 | 1 | 1 | 1 | 1 | 1 |
| | 283 | 0 | 0 | 0 | 0 | 0 |
| | 282 | 0 | 0 | 0 | 0 | 0 |
| | 281 | 0 | 0 | 0 | 0 | 0 |
| 1′-10″ | 280 | 1 | 1 | 1 | 1 | 1 |
| | 279 | 0 | 0 | 0 | 0 | 0 |
| | 278 | 0 | 0 | 0 | 0 | 0 |
| | 277 | 0 | 0 | 0 | 0 | 0 |
| 2′-9″ | 276 | 1 | 1 | 1 | 1 | 1 |
| | 275 | 0 | 0 | 0 | 0 | 0 |
| | 274 | 0 | 0 | 0 | 0 | 0 |
| | 273 | 0 | 0 | 0 | 1 | 0 |
| 3′-8″ | 272 | 1 | 1 | 1 | 0 | 1 |
| | 271 | 0 | 0 | 0 | 0 | 1 |
| | 270 | 0 | 0 | 0 | 1 | 0 |
| | 269 | 0 | 0 | 1 | 0 | 1 |
| 4′-7″ | 268 | 1 | 1 | 0 | 0 | 1 |
| | 267 | 0 | 0 | 0 | 1 | 1 |
| | 266 | 0 | 0 | 1 | 0 | 1 |
| | 265 | 0 | 0 | 0 | 1 | 0 |
| 5′-6″ | 264 | 1 | 1 | 0 | 0 | 1 |
| | 263 | 0 | 0 | 1 | 1 | 1 |
| | 262 | 0 | 0 | 0 | 1 | 0 |
| | 261 | 0 | 1 | 1 | 0 | 1 |
| 6′-5″ | 260 | 1 | 0 | 0 | 1 | 1 |
| | 259 | 0 | 0 | 1 | 1 | 0 |
| | 258 | 0 | 1 | 1 | 0 | 1 |
| | 257 | 1 | 0 | 0 | 1 | 1 |
| 7′-4″ | 256 | 0 | 0 | 1 | 1 | 0 |
| | 255 | 0 | 1 | 1 | 0 | 1 |
| | 254 | 1 | 0 | 0 | 1 | 0 |
| | 253 | 0 | 1 | 1 | 1 | 0 |
| 8′-3″ | 252 | 0 | 0 | 1 | 0 | 1 |
| | 251 | 1 | 1 | 0 | 1 | 0 |
| | 250 | 0 | 1 | 1 | 0 | 0 |
| | 249 | 1 | 0 | 1 | 0 | 1 |
| 9′-2″ | 248 | 0 | 1 | 0 | 1 | 0 |
| | 247 | 1 | 1 | 1 | 0 | 0 |
| | 246 | 1 | 0 | 0 | 0 | 0 |
| | 245 | 0 | 1 | 0 | 0 | 1 |
| 10′-1″ | 244 | 1 | 1 | 0 | 1 | 0 |
| | 243 | 1 | 0 | 1 | 0 | 0 |
| | 242 | 0 | 1 | 0 | 0 | 0 |
| | 241 | 1 | 1 | 0 | 0 | 1 |
| 11′-0″ | 240 | 1 | 0 | 1 | 0 | 1 |
| | 239 | 0 | 1 | 0 | 0 | 0 |
| | 238 | 1 | 0 | 0 | 0 | 1 |
| | 237 | 1 | 0 | 0 | 1 | 0 |
| 11′-11″ | 236 | 0 | 1 | 0 | 1 | 0 |
| No. 9 wire above this line | | | | | | |
| No. 3 wire below this line as indicated | 235 | 1 | 0 | 0 | 0 | 1 |
| | 234 | 0 | 0 | 0 | 1 | 0 |
| | 233 | 0 | 0 | 1 | 0 | 0 |
| 12′-10″ | 232 | 1 | 0 | 1 | 0 | 1 |
| | 231 | 0 | 0 | 0 | 1 | 0 |
| | 230 | 0 | 0 | 0 | 1 | 0 |
| | 229 | 0 | 0 | 1 | 0 | 0 |
| 13′-9″ | 228 | 1 | 0 | 1 | 0 | 0 |
| | 227 | 0 | 0 | 0 | 1 | 0 |
| | 226 | 0 | 0 | 0 | 1 | 0 |
| | 225 | 0 | 0 | 1 | 0 | 1 |
| 14′-8″ | 224 | 1 | 0 | 0 | 1 | 0 |
| | 223 | 0 | 0 | 1 | 0 | 0 |
| | 222 | 0 | 0 | 1 | 0 | 1 |
| | 221 | 0 | 1 | 0 | 0 | 1 |
| 15′-7″ | 220 | 1 | 0 | 0 | 1 | 0 |
| | 219 | 0 | 0 | 1 | 0 | 1 |
| | 218 | 0 | 1 | 0 | 1 | 0 |
| | 217 | 0 | 0 | 1 | 0 | 1 |

| Distance From Top of Silo | Brick Joint | Diameter Inside of Silo 10′ | 12′ | 14′ | 16′ | 18′ |
|---|---|---|---|---|---|---|
| 16′-6″ | 216 | 1 | 0 | 0 | 0 | 0 |
| | 215 | 0 | 1 | 0 | 1 | 0 |
| | 214 | 0 | 0 | 1 | 0 | 1 |
| | 213 | 0 | 0 | 1 | 0 | 1 |
| 17′-5″ | 212 | 1 | 1 | 0 | 1 | 0 |
| | 211 | 0 | 0 | 1 | 0 | 1 |
| | 210 | 0 | 0 | 1 | 0 | 1 |
| | 209 | 0 | 1 | 0 | 1 | 0 |
| 18′-4″ | 208 | 1 | 0 | 1 | 0 | 1 |
| | 207 | 0 | 0 | 1 | 0 | 1 |
| | 206 | 0 | 1 | 0 | 1 | 0 |
| | 205 | 0 | 0 | 1 | 0 | 1 |
| 19′-3″ | 204 | 1 | 0 | 1 | 0 | 1 |
| | 203 | 0 | 1 | 0 | 1 | 0 |
| | 202 | 0 | 0 | 1 | 0 | 1 |
| | 201 | 1 | 1 | 0 | 1 | 0 |
| 20′-2″ | 200 | 0 | 0 | 1 | 0 | 1 |
| | 199 | 0 | 1 | 0 | 1 | 0 |
| | 198 | 1 | 0 | 1 | 0 | 1 |
| | 197 | 0 | 1 | 0 | 1 | 0 |
| 21′-1″ | 196 | 0 | 1 | 0 | 1 | 0 |
| | 195 | 1 | 0 | 1 | 0 | 1 |
| | 194 | 0 | 0 | 1 | 0 | 1 |
| | 193 | 0 | 1 | 0 | 1 | 0 |
| 22′-0″ | 192 | 1 | 0 | 1 | 0 | 1 |
| | 191 | 0 | 0 | 1 | 0 | 1 |
| | 190 | 0 | 1 | 0 | 1 | 0 |
| | 189 | 1 | 0 | 1 | 0 | 1 |
| 22′-11″ | 188 | 0 | 0 | 1 | 0 | 1 |
| | 187 | 0 | 1 | 0 | 1 | 0 |
| | 186 | 1 | 0 | 1 | 0 | 1 |
| | 185 | 0 | 1 | 0 | 1 | 0 |
| 23′-10″ | 184 | 0 | 1 | 0 | 1 | 0 |
| | 183 | 1 | 0 | 1 | 0 | 1 |
| | 182 | 0 | 1 | 0 | 1 | 0 |
| | 181 | 1 | 0 | 1 | 0 | 1 |
| 24′-9″ | 180 | 0 | 1 | 0 | 1 | 0 |
| | 179 | 1 | 0 | 1 | 0 | 1 |
| | 178 | 0 | 1 | 0 | 1 | 0 |
| | 177 | 1 | 0 | 1 | 0 | 1 |
| 25′-8″ | 176 | 0 | 1 | 0 | 1 | 0 |
| | 175 | 1 | 0 | 1 | 0 | 1 |
| | 174 | 0 | 1 | 0 | 1 | 0 |
| | 173 | 1 | 0 | 1 | 0 | 1 |
| 26′-7″ | 172 | 0 | 1 | 0 | 1 | 0 |
| | 171 | 1 | 0 | 1 | 0 | 1 |
| | 170 | 0 | 1 | 0 | 1 | 0 |
| | 169 | 1 | 0 | 1 | 0 | 1 |
| 27′-6″ | 168 | 0 | 1 | 0 | 1 | 0 |
| | 167 | 1 | 0 | 1 | 0 | 1 |
| | 166 | 0 | 1 | 0 | 1 | 0 |
| | 165 | 1 | 0 | 1 | 0 | 1 |
| 28′-5″ | 164 | 0 | 1 | 0 | 1 | 0 |
| | 163 | 1 | 0 | 1 | 1 | 1 |
| | 162 | 0 | 1 | 0 | 1 | 0 |
| | 161 | 1 | 0 | 1 | 0 | 1 |
| 29′-4″ | 160 | 0 | 1 | 0 | 1 | 0 |
| | 159 | 1 | 0 | 1 | 0 | 1 |
| | 158 | 1 | 0 | 1 | 0 | 1 |
| | 157 | 1 | 0 | 1 | 0 | 1 |
| 30′-3″ | 156 | 0 | 1 | 0 | 1 | 0 |
| | 155 | 1 | 0 | 1 | 0 | 1 |
| | 154 | 0 | 1 | 0 | 1 | 0 |
| | 153 | 1 | 0 | 1 | 0 | 1 |
| 31′-2″ | 152 | 0 | 1 | 0 | 1 | 0 |
| | 151 | 1 | 0 | 1 | 1 | 1 |
| | 150 | 0 | 1 | 0 | 1 | 0 |
| | 149 | 1 | 0 | 1 | 0 | 1 |
| 32′-1″ | 148 | 0 | 1 | 0 | 1 | 0 |
| | 147 | 1 | 0 | 1 | 0 | 1 |
| | 146 | 0 | 1 | 0 | 1 | 0 |
| | 145 | 1 | 0 | 1 | 0 | 1 |
| No. 3 wire above this line as indicated | | | | | | |
| ⅜″ round rods below this line | | | | | | |

## TABLE NO. 60

## TABLE OF STEEL REINFORCEMENT FOR REINFORCED BRICK MASONRY SILOS—Continued

**Indicating Number of No. 9 Wires, No. 3 Wires, or ⅜″ Round Rods per Joint**

**One ⅜″ Round Rod may be Considered Equal to Two No. 3 Wires or Equal to Six No. 9 Wires**

| Distance From Top of Silo | Brick Joint | Diameter Inside of Silo | | | | | | Distance From Top of Silo | Brick Joint | Diameter Inside of Silo | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 10′ | 12′ | 14′ | 16′ | 18′ | | | | 10′ | 12′ | 14′ | 16′ | 18′ |
| 33′-0″ | 144 | 0 | 1 | 0 | 0 | 0 | | 49′-6″ | 72 | 0 | 0 | 0 | 0 | 1 |
| | 143 | 1 | 1 | 1 | 1 | 1 | | | 71 | 1 | 1 | 1 | 1 | 1 |
| | 142 | 0 | 0 | 1 | 0 | 0 | | | 70 | 0 | 0 | 0 | 1 | 0 |
| | 141 | 1 | 1 | 1 | 1 | 1 | | | 69 | 1 | 1 | 1 | 0 | 1 |
| 33′-11″ | 140 | 0 | 1 | 0 | 0 | 0 | | 50′-5″ | 68 | 0 | 0 | 1 | 0 | 1 |
| | 139 | 1 | 0 | 1 | 1 | 1 | | | 67 | 1 | 1 | 1 | 1 | 0 |
| | 138 | 0 | 1 | 0 | 0 | 0 | | | 66 | 0 | 0 | 1 | 0 | 1 |
| | 137 | 1 | 1 | 1 | 1 | 1 | | | 65 | 1 | 1 | 0 | 1 | 1 |
| 34′-10″ | 136 | 0 | 0 | 0 | 0 | 0 | | 51′-4″ | 64 | 0 | 0 | 1 | 1 | 0 |
| | 135 | 1 | 1 | 1 | 1 | 1 | | | 63 | 1 | 1 | 1 | 0 | 1 |
| | 134 | 0 | 1 | 0 | 0 | 0 | | | 62 | 0 | 0 | 0 | 1 | 1 |
| | 133 | 1 | 0 | 1 | 1 | 1 | | | 61 | 1 | 1 | 1 | 1 | 0 |
| 35′-9″ | 132 | 0 | 1 | 0 | 0 | 0 | | 52′-3″ | 60 | 0 | 0 | 1 | 0 | 1 |
| | 131 | 1 | 1 | 1 | 1 | 1 | | | 59 | 1 | 1 | 0 | 1 | 1 |
| | 130 | 0 | 0 | 0 | 0 | 0 | | | 58 | 0 | 0 | 1 | 1 | 0 |
| | 129 | 1 | 1 | 1 | 1 | 1 | | | 57 | 1 | 1 | 1 | 0 | 1 |
| 36′-8″ | 128 | 0 | 1 | 0 | 0 | 0 | | 53′-2″ | 56 | 0 | 0 | 0 | 1 | 1 |
| | 127 | 1 | 0 | 1 | 1 | 1 | | | 55 | 1 | 1 | 1 | 1 | 0 |
| | 126 | 1 | 1 | 0 | 0 | 0 | | | 54 | 0 | 0 | 1 | 0 | 1 |
| | 125 | 0 | 1 | 1 | 1 | 1 | | | 53 | 1 | 1 | 0 | 1 | 1 |
| 37′-7″ | 124 | 1 | 0 | 0 | 0 | 0 | | 54′-1″ | 52 | 0 | 0 | 1 | 1 | 0 |
| | 123 | 1 | 1 | 1 | 1 | 1 | | | 51 | 1 | 1 | 1 | 0 | 1 |
| | 122 | 0 | 1 | 0 | 0 | 0 | | | 50 | 0 | 0 | 0 | 1 | 1 |
| | 121 | 1 | 1 | 1 | 1 | 1 | | | 49 | 1 | 1 | 1 | 1 | 0 |
| 38′-6″ | 120 | 1 | 0 | 0 | 0 | 0 | | 55′-0″ | 48 | 1 | 0 | 1 | 0 | 1 |
| | 119 | 0 | 1 | 1 | 1 | 1 | | | 47 | . | 1 | 0 | 1 | 1 |
| | 118 | 1 | 1 | 0 | 0 | 0 | | | 46 | . | 0 | 1 | 1 | 0 |
| | 117 | 1 | 1 | 1 | 1 | 1 | | | 45 | . | 1 | 1 | 0 | 1 |
| 39′-5″ | 116 | 0 | 0 | 0 | 0 | 0 | | 55′-11″ | 44 | . | 0 | 0 | 1 | 1 |
| | 115 | 1 | 1 | 1 | 1 | 1 | | | 43 | . | 1 | 1 | 0 | 1 |
| | 114 | 1 | 1 | 0 | 0 | 0 | | | 42 | . | 0 | 1 | 0 | 1 |
| | 113 | 0 | 1 | 1 | 1 | 1 | | | 41 | . | 1 | 0 | 1 | 1 |
| 40′-4″ | 112 | 1 | 0 | 0 | 0 | 0 | | 56′-10″ | 40 | . | 0 | 1 | 1 | 0 |
| | 111 | 1 | 1 | 1 | 1 | 1 | | | 39 | . | 1 | 1 | 0 | 1 |
| | 110 | 0 | 0 | 0 | 0 | 1 | | | 38 | . | 0 | 0 | 1 | 1 |
| | 109 | 1 | 1 | 1 | 1 | 0 | | | 37 | . | 1 | 1 | 1 | 0 |
| 41′-3″ | 108 | 1 | 0 | 0 | 0 | 1 | | 57′-9″ | 36 | . | 0 | 1 | 0 | 1 |
| | 107 | 0 | 1 | 1 | 1 | 1 | | | 35 | . | 1 | 0 | 1 | 1 |
| | 106 | 1 | 0 | 0 | 0 | 0 | | | 34 | . | 0 | 1 | 1 | 0 |
| | 105 | 1 | 1 | 1 | 1 | 1 | | | 33 | . | 1 | 1 | 0 | 1 |
| 42′-2″ | 104 | 0 | 0 | 0 | 0 | 1 | | 58′-8″ | 32 | . | 0 | 0 | 1 | 1 |
| | 103 | 1 | 1 | 1 | 1 | 0 | | | 31 | . | 1 | 1 | 1 | 0 |
| | 102 | 1 | 0 | 0 | 0 | 1 | | | 30 | . | 0 | 1 | 0 | 1 |
| | 101 | 0 | 1 | 1 | 1 | 1 | | | 29 | . | 1 | 0 | 1 | 1 |
| 43′-1″ | 100 | 1 | 0 | 0 | 0 | 0 | | 59′-7″ | 28 | . | 0 | 1 | 1 | 0 |
| | 99 | 1 | 1 | 1 | 1 | 1 | | | 27 | . | 1 | 1 | 0 | 1 |
| | 98 | 0 | 0 | 0 | 0 | 1 | | | 26 | . | 0 | 0 | 1 | 1 |
| | 97 | 1 | 1 | 1 | 1 | 0 | | | 25 | . | 1 | 1 | 1 | 0 |
| 44′-0″ | 96 | 1 | 0 | 0 | 0 | 1 | | 60′-6″ | 24 | . | 0 | 1 | 0 | 1 |
| | 95 | 0 | 1 | 1 | 1 | 1 | | | 23 | . | 1 | 0 | 1 | 1 |
| | 94 | 1 | 0 | 0 | 1 | 0 | | | 22 | . | 0 | 1 | 1 | 0 |
| | 93 | 1 | 1 | 1 | 0 | 1 | | | 21 | . | 1 | 1 | 0 | 1 |
| 44′-11″ | 92 | 0 | 0 | 0 | 1 | 1 | | 61′-5″ | 20 | . | 0 | 0 | 1 | 1 |
| | 91 | 1 | 1 | 1 | 1 | 0 | | | 19 | . | 1 | 1 | 1 | 0 |
| | 90 | 1 | 0 | 0 | 0 | 1 | | | 18 | . | 0 | 1 | 0 | 1 |
| | 89 | 0 | 1 | 1 | 1 | 1 | | | 17 | . | 1 | 0 | 1 | 1 |
| 45′-10″ | 88 | 1 | 0 | 0 | 1 | 0 | | 62′-4″ | 16 | . | 0 | 1 | 1 | 0 |
| | 87 | 1 | 1 | 1 | 0 | 1 | | | 15 | . | 1 | 1 | 0 | 1 |
| | 86 | 0 | 0 | 0 | 1 | 1 | | | 14 | . | 0 | 0 | 1 | 1 |
| | 85 | 1 | 1 | 1 | 1 | 0 | | | 13 | . | 1 | 1 | 1 | 1 |
| 46′-9″ | 84 | 1 | 0 | 0 | 0 | 1 | | 63′-3″ | 12 | . | 0 | 1 | 0 | 1 |
| | 83 | 0 | 1 | 1 | 1 | 1 | | | 11 | . | 1 | 0 | 1 | 1 |
| | 82 | 1 | 0 | 0 | 1 | 0 | | | 10 | . | 0 | 1 | 1 | 1 |
| | 81 | 1 | 1 | 1 | 0 | 1 | | | 9 | . | 1 | 1 | 0 | 1 |
| 47′-8″ | 80 | 0 | 0 | 0 | 1 | 1 | | 64′-2″ | 8 | . | 0 | 0 | 1 | 1 |
| | 79 | 1 | 1 | 1 | 1 | 0 | | | 7 | . | 1 | 1 | 1 | 1 |
| | 78 | 1 | 0 | 0 | 0 | 1 | | | 6 | . | 0 | 1 | 0 | 1 |
| | 77 | 1 | 1 | 1 | 1 | 1 | | | 5 | . | 1 | 0 | 1 | 1 |
| 48′-7″ | 76 | 0 | 0 | 0 | 1 | 0 | | 65′-1″ | 4 | . | 0 | 1 | 1 | 1 |
| | 75 | 1 | 1 | 1 | 0 | 1 | | | 3 | . | 1 | 1 | 0 | 1 |
| | 74 | 1 | 0 | 0 | 1 | 1 | | | 2 | . | 0 | 0 | 1 | 1 |
| | 73 | 1 | 1 | 1 | 1 | 0 | | 66′-0″ | 1 | . | 1 | 1 | 0 | 1 |
| | | | | | | | | | 0 | . | . | . | . | . |

No. 3 wire above

⅜″ round rods below

Reinforcements in this table are ⅜″ round rods

has less elasticity than the steel, it will of necessity crack. In the average case, there will be a large number of very small cracks which may not be detrimental to the silo. However, it is possible that one or more cracks may be of sufficient width to cause leakage and will have to be pointed up. A number of silos have been constructed in which the unit stress in the steel is figured at 30,000 pounds per square inch. However, we do not recommend this high stress, and, since the cost of the reinforcing steel is a minor item in the cost of the silo, it would seem poor economy to run the risk of cracked masonry through the reduction of the steel area.

The following table of steel requirements for a brick masonry silo is based upon an allowable stress in the steel of approximately 16,000 pounds per square inch.

The area of steel required in the 1 foot section of the example stated above, is:

$$A_s = \frac{2904}{16{,}000} = 0.182 \text{ sq. in.}$$

⅜″ round bars placed on 7¼″ centers would satisfy this requirement. It will be necessary, however, to place the bars in mortar joints which are 2¼″ on centers. The available spacing is in every second joint (5½″ centers) or in every third joint (8¼″ centers). It is apparent that the spacing provided by every third joint will not satisfy the required area of reinforcing steel. Therefore, the reinforcement in a 1 ft. section, 44 feet below the top of the silo, will consist of ⅜″ bars in every other mortar joint. By referring to the Table of Steel Reinforcement for R-B-M Silo, it is noted that the reinforcement for a silo of 12 ft. diameter at 44 ft. below the top is a ⅜″ round rod in every other joint.

Table 59 shows the number and size of the reinforcing wires or rods which should be imbedded in each joint of silos of various heights. In numbering joints, start with the first course above the footing. For a 66 ft. silo, the joint between the top of the footing (8″ wide) and the 4″ wall would be numbered zero. The joint between the first and second courses of the 4″ wall would be No. 1 and so on to the top of the silo.

As an example of the use of the table, assume that it is desired to construct a silo 16 ft. in diameter and 55 ft. high. The joint between the footing and the 4″ wall then becomes No. 48 and, from the table, no reinforcement is required in this joint. The joint between the first and second courses is Joint 49 and the steel requirement for this joint is a ⅜″ rod.

Number the joints upward as the work progresses and include in each joint the reinforcement specified in the table. The location of any joint in the table may be easily obtained by counting the joints above the footing and adding this number to 48.

## 1505. REINFORCED BRICK MASONRY IN BRIDGES

### (a) Ohio Highway Bridge

Mr. J. R. Burkey, Chief Engineer of Bridges, Ohio Department of Highways, reported in Engineering News Record, Feb. 28, 1935, that a highway bridge of unusual type, in that it made rather extensive use of reinforced brick masonry, has recently been opened to traffic by the department of highways of Ohio. The structure is located on state route 39 at the east corporation line of the village of Sugar Creek, Tuscarawas County, and spans a small stream known as the South Fork of Sugar Creek.

The available foundation soil was unsuitable for brick arch construction, a twin-span concrete T-beam super-structure, on a reinforced brick masonry substructure and with brick facing and railing, was chosen after preliminary estimates indicated this type to be economically justified.

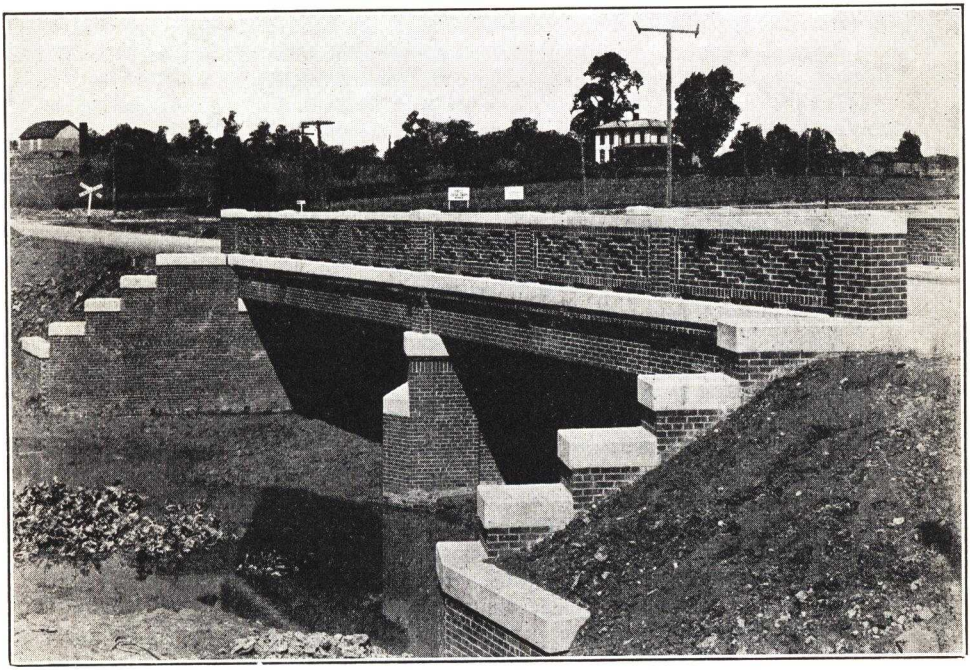
The two spans, skewed 22 deg., are each 32 ft. clear, and accommodate a 24-ft. roadway and two 3-ft. refuge walks. The height from bottom of footings to top of curb varies somewhat for the different substructure units, but is approximately 20 ft.; the footings in all cases consisting of concrete resting on timber piles. Above the footings

all pier and abutment walls are made of reinforced brick masonry capped with concrete, to protect the vertical joints of the brickwork against entrance of water and to produce a more finished appearance.

In line with the general policy of the bureau of bridges, the abutments are designed with breast wall and wings in a straight line, to facilitate the future widening of the bridge.

Details for the structure were prepared for the use of the standard-size building brick 2¼ x 3¾ x 8 in. with permission to use any size of building or paving brick, if details showing variations in dimensions and patterns were presented by the contractor and approved by the bureau of bridges. Face brick was required to be smooth, dark red and hard-burned, meeting A.S.T.M. specification No. C62-30 for hard or medium grade building brick. Backing brick was required to meet specifications C34-24 for Class A or B vitrified, or hard or medium grade clay sewer brick. Face brick was also required to meet the absorption test for hard-grade sewer brick. The brick used was standard-size, dark red, building brick. Common bond was used, with every sixth course as headers, and joints were ½ in. thick. All joints were completely filled with mortar, and all exposed joints were struck, including those on the back of the abutment walls.

For experimental purposes, so as to determine relative efflorescence, workability, durability, etc. two types of mortars were used. The east abutment was built with 1:3 portland cement mortar to which was added hydrated lime, not to exceed 15 per cent by volume of the cement. The remainder of the brick masonry was built with 1:2½ mortar, using a commercial mortar cement. The latter mortar proved to be much more workable and easy to handle. Its crushing strength, although lower than the portland cement mortar, was sufficiently high to provide a reasonable factor of safety over the design stresses.



Waterproofing, consisting of two coats of hot asphalt, was applied to the back of all abutment brickwork below the ground line; total of 1 gal. per sq. yd. of surface was used. This, together with the concrete capping on all brickwork, was expected to reduce efflorescence to a minimum.

The sidewalks were built as reinforced brick slabs, spanning from curb to coping. Each slab was 3¾ in. (one brick on edge) thick and reinforced with a ⅜ in. round rod in each joint (2¾ in. centers). This slab was precast or laid up as a 4 in. wall in sections about 2 ft. long, allowed to season, and then placed on a mortar bed at both ends. The sidewalks are so designed that they may be removed to facilitate widening of the roadway if required.

An H-15 design loading was used with the following unit working stress in the reinforced brick masonry walls, railing and sidewalk slab; compressive stress in brick masonry, 600 lb. per sq. in.; shear in brick masonry, 20 lb. per sq. in.; bond for deformed bars, 50 lb. per sq. in.; tensile stress in reinforcing steel, 14,000 lb. per sq. in.; and in ratio of modulus of elasticity of steel to brick masonry of 15.

The superstructure was not continuous, but both spans were anchored to the pier, with the abutment ends of each beam resting on two ½-in. rolled phosphor-bronze plates, separated by one layer of 1/16-in. sheet asbestos packing and flake graphite. An open joint 1½ in. wide was left between the sides of the superstructure and the abutment, and a waterproof expansion joint of folded copper and premolded expansion joint filler was provided at the ends of the roadway slab.

Brick portions of the structure were built in a shorter time than if concrete had been used. The cost of the reinforced brick masonry in the substructure was $13.65 per cu. yd., which compares quite favorably with the usual cost of pier and abutment masonry, while the bid price of $4.00 per linear foot for the railing is in line with bids received for other durable railings in which architectural treatment is given prominence. The contract price for the structure was $16,150.00, not including piling.

Although this design was developed somewhat in the spirit of research, with the added inducement of maximum local employment, the finished structure has received such favorable comment generally that this type of construction can be readily justified whenever conditions and availability of materials are favorable.

The bridge was built by O'Connor, Kipf Co., Cleveland. The work was done under the direction of O. W. Merrell, as Director of Highways; H. P. Chapman, Assistant Director; J. R. Burkey, Chief Engineer of Bridges; and Elmer Hilty, Chief Engineer of Construction. J. C. Merrell, of the Bureau of Bridges, developed the details of the design.

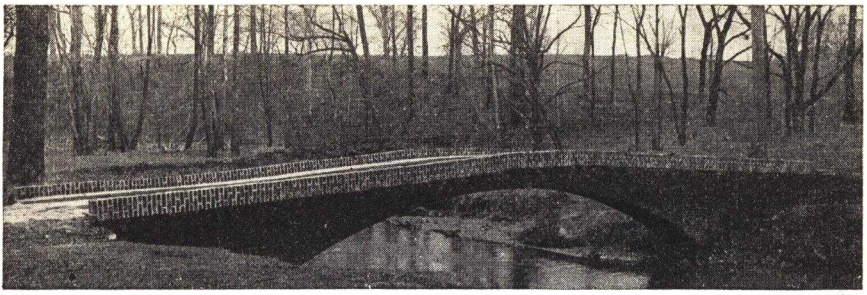
**(b) R-B-M Arch Bridge Near Philadelphia**

On the grounds of the Melrose Country Club near Philadelphia, a 42-foot span arch bridge of reinforced brick masonry, furnished another example of the possibilities of this type of brick construction. The bridge was designed and its erection supervised by Judson Vogdes, Consulting Engineer of Philadelphia, who has contributed a great deal towards a better understanding and fuller appreciation of reinforced brick masonry.

The astonishing feature of this particular operation was that it was built entirely without the use of false work. Abutments were built in the banks on either side of the stream it spans and from these, the work proceeded by cantilevering outward; the brick being laid up on short, movable centers, suspended from the cantilever arms.

The bridge was designed to accommodate the tractor-drawn mowing machines used in the upkeep of the grounds. These tractors weigh 4,000 pounds and shortly after the bridge was completed, it was tested for strength. It supported two tractors, two trucks and an automobile, together with their drivers, with no visible evidence of strain at any point.

The barrel of the arch has an overall width of four feet. At each edge are 8-inch spandrel walls, which rise to support pre-cast reinforced brick deck slabs 12 feet long.



Along each side of the deck are built reinforced guards to the height of eight inches. This allows a net roadway of a little more than 10½ feet. The arch has a rise of six feet. The reinforcement consisted of one-half and three-eighth inch diameter steel rods.

First consideration in its designing was given to strength and width. The four-foot-wide tractors, each weighing 4,000 pounds, offered the heaviest loads it would probably be called upon to carry. The mowing machines required a road clearance of 10 feet. So the bridge was designed to carry a live load of 80 pounds to the square foot over its entire deck.

The 4,000 pound tractors, distributing their weight on four wheels, will be run directly over the central spandrel walls. The mowers are comparatively light and their weight also will be very largely centered. For all other traffic, including automobiles, the floor strength will be ample.

For the cantilever construction, a mortar consisting of two parts high-early-strength portland cement, one part natural cement and one part sand was used. This unusual mixture proved very satisfactory and afforded a strength at the end of 24 hours sufficient to allow the use of the work under load. The natural cement was used solely to increase the plasticity and workability of the mortar. In constructing the pre-cast slabs straight portland cement and lime were used, mixed 1½:1:6.

Results of earlier slab tests largely dictated the deck slab design. The reinforced curb guards along each side of the deck served both to tie together the deck slabs and to increase their rigidity. Stirrup rods in the spandrel walls tied the arch ring to the tension side of the anchor beams during erection, but their usefulness ceased when the juncture of the cantilever arms completed the arch and left only compressive stresses.

Abutment caissons, 6 x 8 feet and 2½ feet deep, reinforced with half-inch rods and sunk to gravel by excavating inside, formed the bases from which the cantilevering extended. On these abutments, the land sections of the work were built with reinforced spandrel walls two feet deep and eight inches thick extending shoreward 12 feet as counter-balances for the half arches. On these walls the floor slabs were immediately laid to insure the efficiency of the counterweights. The plan was entirely successful.

No difficulty was experienced in the arch construction. From either cantilever arm was extended a movable wood center, probably 10 feet long, and from this the masons worked in building up four-foot sections of the arch ring and spandrel walls. Two days were required to complete such a section. With each arm completed to a length of 17 feet, one center was discarded and the other suspended to span the breach and allow the making of the final closure.

Placing of the pre-cast deck slabs began as soon as the arch was finished. They were attached to the spandrel walls by a mortar bond and to each other by grouting in the half-inch joint left between each of them, a form board being placed under each joint. Thirty-seven of these slabs were used on the bridge and its approaches.

In anticipation of the curb, the floor slabs were built with toothed ends and in the openings, so provided, soldier courses were set, supported by a form board hung under

the slabs. These soldier brick were bonded to the slabs by mortar pointing and grouting, providing a strong and thoroughly efficient base for the double reinforced curb beam.

Twenty thousand brick were used in the structure, which was built with surprising speed, considering that it was the first of its kind, so far as anyone seems to know, in more than a century. Twelve days of bricklayer time were consumed on the arch and spandrel walls. During this time another mason built the pre-cast deck slabs.

## 1506. BRICK FORMS FOR CONCRETE

Brickwork has been used as a "form" for concrete masonry for many years, but its advantages seem not to have been generally recognized, and its use for this purpose not greatly extended. Only a few of our designers have so used brickwork, but in several foreign countries it is quite common practice.

Perhaps the most notable example in this country is the Caneadea dam on Caneadea Creek, New York, built for the Rochester Gas & Electric Corp. by Gannett, Seelye and Flemming, Inc., of Harrisburg, Pa., and designed by this same noted firm of engineer-constructors.

Both faces of this arch dam, 600 feet long and 143 feet high above grade, are faced with brickwork 8 inches thick. This brickwork, of alternate headers and stretchers, was built in advance of the concrete pouring and used as forms for this work. But, according to a statement of the designers, a major function of the brickwork is to protect the concrete from the ravages of frost action and other disintegrating forces.

The effective and economical use of brickwork as a form for concrete is, of course, in masonry sections which are vertical, or nearly so.

In this case, the brickwork needs no support, is stable as the work proceeds, and can be readily built in advance of the concrete pouring. In this country, these considerations have influenced its use as the facing of retaining walls and similar structures.

For most all types of construction, a 4-inch "form" or facing of brick is sufficient. If we assume that a brick mason lays only 600 or 700 brick per day (the production should be much higher) at a cost of $25 per thousand, this is the equivalent of 15.4 cents per square foot of wall. This is just about the amount usually figured for wood forms, so that the labor cost of the brickwork just about balances the cost of wood forms. If brick cost $15 per thousand, this is the equivalent of 9.25 cents per sq. ft. of 4-inch wall. If concrete cost is $8.50 per cubic yard, then the equivalent cost of the concrete displaced (4 inches thick) is 10.5 cents. Therefore, the actual construction cost of the brickwork is less than the cost of an equal volume of concrete plus the cost of wooden forms. A fair conclusion is that where it is practical to use this type of construction, a concrete wall faced with 4 inches of brickwork may be built for the same or less money than a plain concrete wall using wooden forms, but this is not the only advantage. If the concrete has to be finished after the forms are stripped, then the brick-faced wall has a decided advantage, for the brick facing is a finished surface and may be a very attractive one.

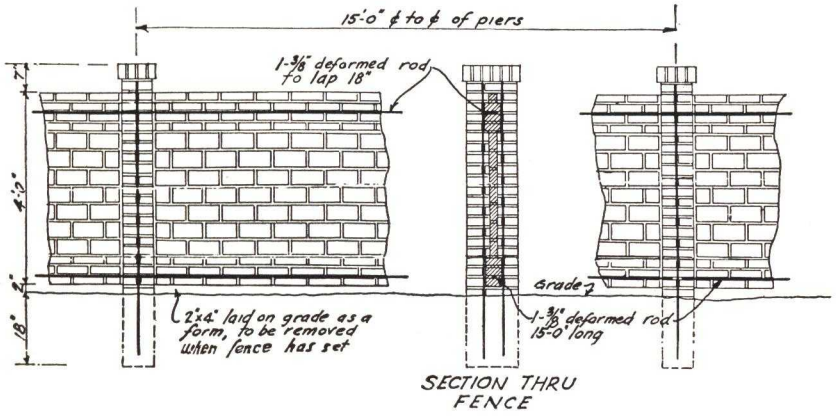
For massive walls, or massive parts of structures, it may be necessary to use an 8-inch thickness of brick facing, as was done in the Caneadea dam, but even in this case the finished cost should still be in favor of such construction and the advantages mentioned above would still apply.

## 1507. A REINFORCED BRICK MASONRY FENCE

When one mentions a brick fence, the first impression is that of a garden wall or similar structure, where appearance and permanence is of primary importance and cost, secondary. The fence described herein is typical of the possibilities of R-B-M construction. It retains the beauty, charm and permanence of the brick wall and in addition, actually competes favorably in cost with many of the ordinary type of fences.

Located on the main highway between Camden and New York, this reinforced brick masonry fence partly encloses the Crescent Burial Park. This fence was another one of the many R-B-M structures designed by Judson Vogdes, Consulting Engineer of Philadelphia. The structure consists of piers or posts located every fifteen feet, between which are built deep beams standing free from the ground. The height of the wall or fence is four feet with six-inch projections at the piers. By using reinforcing bars in both piers and beams, the amount of brick required was reduced and also the cost.

Every fifteen feet there is a vertically reinforced pier. These are 8 inches by 12 inches with the 12-inch dimension transverse to the beams. Thus with the 4-inch beams in the center, a 4-inch pilaster is formed on each side of the fence. A rectangular excavation was made to a depth of 18 inches slightly larger than the pier. On the bottom of this a rowlock course was laid on a thin bed of mortar and two vertical reinforcing bars placed. These bars were then inclosed in the pier which was brought up to a height as set by the proposed future grade. In some cases, the pier was carried up six or eight inches above the level of the present grade. The piers were built plumb, while the beams are conforming to the grade of the street. The usual method of building was to carry up four or five courses in the piers and then fill in the beams between.



SECTION THRU FENCE

In the beams which form the body of the fence there are some interesting problems of design. Not only did the fence have to span the fifteen feet between piers supporting its own weight and a live load, but it also had to resist wind and other horizontal pressures from either side. With a member only three and three-quarters inches thick on a fifteen-foot span this presented a unique problem. To begin with, the cost of the fence was definitely limited, which meant the use of every economy possible consistent with good design. The design finally adopted consisted of a four-foot deep beam built with three flat courses at the top and bottom and seven rowlock courses between. Thus the top and bottom of the beams, where the reinforcing bars were placed, were one brick thick and the remaining portion only two and one-quarter inches thick. The piers and flat courses formed a frame around the flat courses and stiffened these latter. Without the use of reinforcing, such a design would not be safe even if the fence rested on the ground. In spite of the relatively thin wall and the long span, the entire structure is remarkably stiff.

The brick used in the fence were hard-burned manufactured at Maple Shade, N. J. Mortar was composed of one part portland cement and three parts clean washed sand to which was added fifteen pounds of Diataid per bag of cement. This latter material is

an admixture used in place of lime to render the mortar plastic and workable without the use of excess water.

Although the design used in this fence is a somewhat radical departure from usual practice, there was considerable background to justify its use. For some time the construction of four-inch reinforced brick garage walls has been in use in New Jersey. The fence is only a step further. The information gained from the testing of demonstration structures of reinforced brick masonry aided materially in making the design used.

This fence shows that brick can be used for such a purpose at a cost far below that considered possible. In fact, this particular type of fence is being quoted in place at one dollar per lineal foot.

## 1508. SWIMMING POOLS

The cleanliness and beauty of a swimming pool are two of the main features that impress the bathers. Practically all of the swimming pools provide this feature by the use of smooth, hard-burned brick or tile, vitrified brick or glazed brick and tile. Such surfaces are clean and sanitary, and resist the growth of algea.

In many instances, these materials are applied as a veneer lining to the structural material. By use of reinforced brick masonry in structural design of swimming pools, it is possible to include these materials as part of the structural wall, effecting not only a saving but obtaining also a better structural design. There are many of the reinforced brick masonry type of swimming pools in private estates, clubs and institutions, but a typical example of the use of reinforced brick masonry for outdoor swimming pools was constructed by the City of Springfield, Minn. in 1936.

This pool was sponsored by the American Legion and built with the assistance of the Works Progress Administration. The architects were LeRoy Daulbert and Malcolm Allen, of Ames, Iowa. The pool is 50 x 120 ft. in size, 2 ft. 6 in. deep at the shallow end and 9 ft. 6 in. deep under the diving board. Around the pool are brick walls 8 ft. wide at the sides and 15 and 20 ft. wide at the ends. The project included an efficient, and adequate filtration and purification equipment and a spacious bath-house with ladies' dressing room, men's dressing room and check room.

The footings were of reinforced concrete. The side walls were 13 inches thick, consisting of a facing of smooth brick, one inch of mortar and eight inches of load-bearing double-shell tile. Vertically reinforced pilasters strengthened the wall at intervals of every 12 feet, the side walls being designed as a continuous member and reinforced horizontally in the mortar joints between the tile and vertically between the brick and tile. Horizontal reinforcing was bent into the pilasters, which were designed as cantilevers supporting the walls.

Two courses of brick were laid in the reinforced floor. The lower was laid flat with a shoved joint on which was poured a half-inch layer of asphalt. The top course was then laid on a mortar bed, in basket weave pattern. Tempered steel reinforcing was placed 8½ inches on center across, and 17½ inches on center the length of the pool. Walks are a single course of brick in running bond, reinforced with ¼-inch round bars 8½ inches on center the width of the walks.

The mortar consisted of one part portland cement, three parts sand, ⅓ part finely-ground clay admixture, and one quart of Omicron waterproofing. Great care was taken to insure full head joints in the brick and tile work and to securely bed the steel. The walls were kept sprinkled for seven days after their construction was finished. Materials used included 88,000 face and 60,000 common brick; 6,000 face, 7,000 smooth and some 5,500 other type tile for roof, beams and partitions.

The total cost of this swimming pool, including pumping and filter equipment, fence, diving boards, etc., was $22,648.15. The cost of brick, tile, mortar and reinforcing materials was less than 20% of this total.

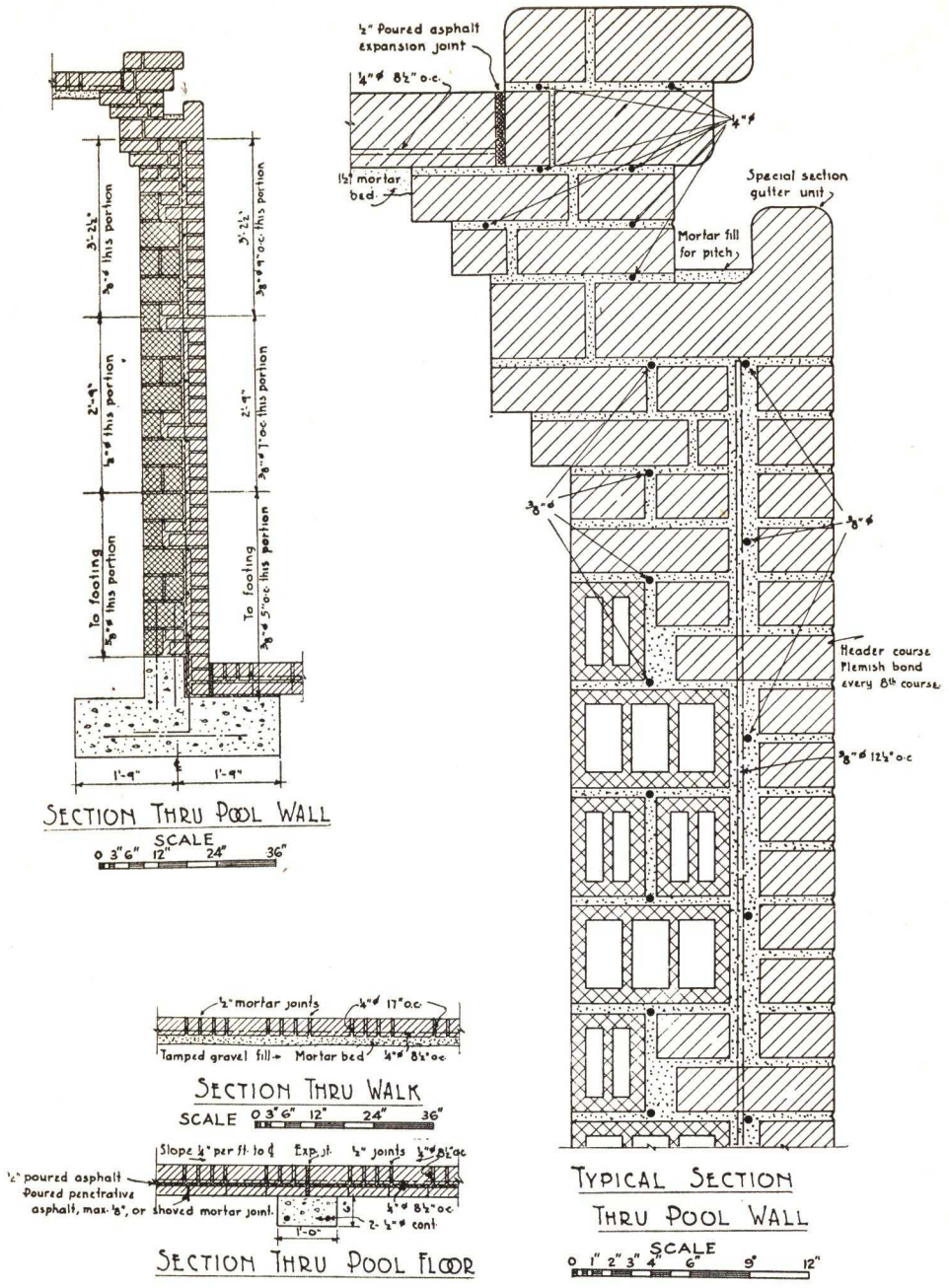



*Figure 43*

*Sections of swimming pool walls, floor and walk*

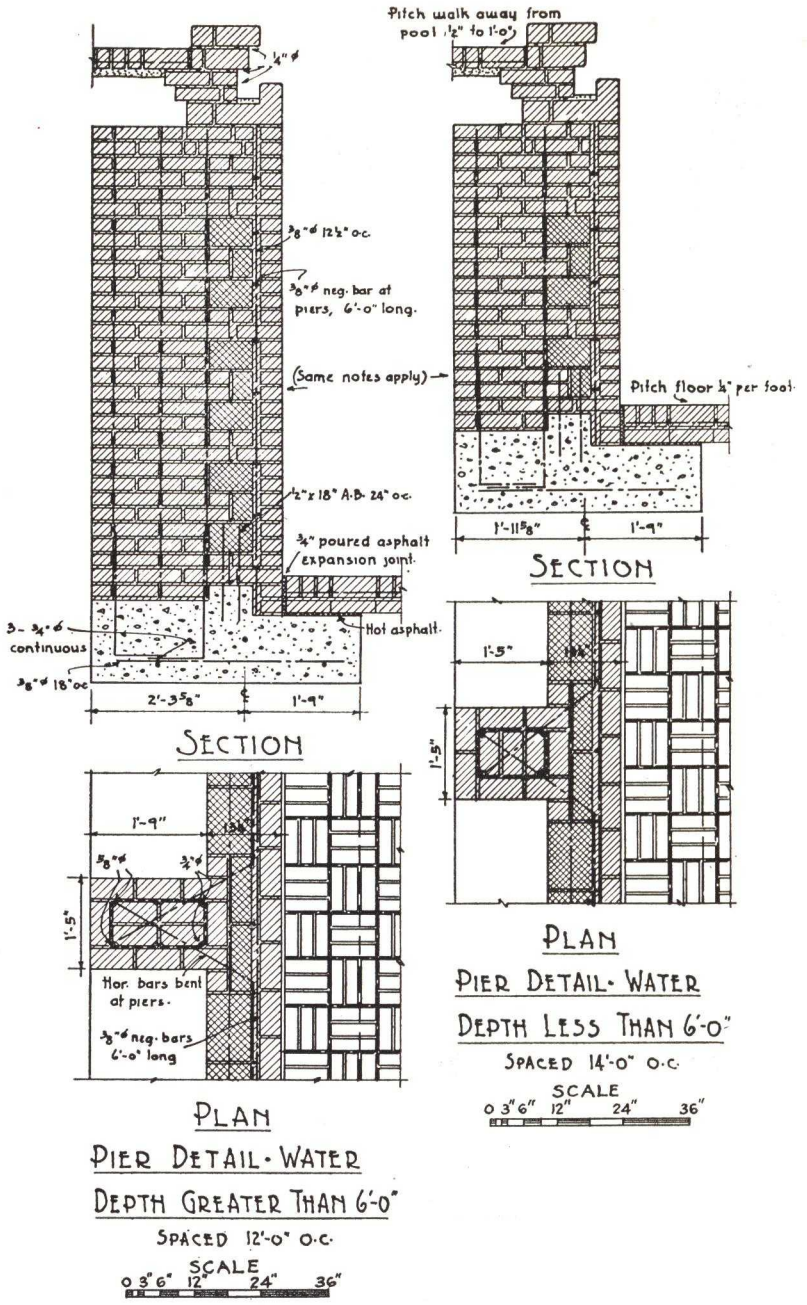



*Figure 44*

*Elevation and plan sections of swimming pool wall at typical piers*

# APPENDIX

The following table gives the number of brick and amount of mortar required in brick walls, assuming the standard size of brick 3¾ x 2¼ x 8 inches. Three widths of mortar joints are given, ⅜", ½" and ⅝". The quantities are for walls having a thickness of one brick in various positions. In estimating walls composed of more than one wythe of brick, the quantity of brick is obtained by multiplying the number in one wythe by the number of wythes, and the amount of mortar for one wythe is likewise multiplied by the number of wythes, to which is also added the mortar comprising the interior vertical joint between the wythes. An interior vertical joint is usually ½" thick and amounts to 0.0417 cubic ft. of mortar per square foot of wall area.

## TABLE NO. 61
## ESTIMATING BRICK AND MORTAR QUANTITIES

| Standard Brick Size 3¾x2¼x8 | Wythe Thickness | Face Area of Unit in Sq. Ft. | No. Brick per ft. Height | No. Brick per ft. Length | No. Brick per sq. ft. | Brick Courses in Height | Cu. Ft. Mortar per Brick |
|---|---|---|---|---|---|---|---|
| | | Including ⅜" visible mortar joints — Add ½" interior vertical mortar | | | | | |
| (a) Stretcher, edge | 2¼" | .2399 | 2.908 | 1.433 | 4.168 | 1.571 | .0059 |
| (b) Stretcher, flat | 3¾" | .1527 | 4.571 | 1.433 | 6.562 | 1.000 | .0086 |
| (c) Header, flat | 8" | .0752 | 4.571 | 2.908 | 13.298 | 1.000 | .0110 |
| (d) Rowlock, edge | 8" | .0752 | 2.908 | 4.570 | 13.298 | 1.571 | .0110 |
| (e) Soldier, end | 3¾" | .1527 | 1.433 | 4.570 | 6.562 | 3.189 | .0086 |
| | | Including ½" visible mortar joints — Add ½" interior vertical mortar | | | | | |
| (a) Stretcher, edge | 2¼" | .2509 | 2.823 | 1.410 | 3.986 | 1.545 | .0079 |
| (b) Stretcher, flat | 3¾" | .1623 | 4.363 | 1.410 | 6.161 | 1.000 | .0117 |
| (c) Header, flat | 8" | .0812 | 4.363 | 2.823 | 12.318 | 1.000 | .0149 |
| (d) Rowlock, edge | 8" | .0812 | 2.823 | 4.363 | 12.318 | 1.545 | .0150 |
| (e) Soldier, end | 3¾" | .1623 | 1.411 | 4.363 | 6.161 | 3.090 | .0117 |
| | | Including ⅝" visible mortar joints — Add ½" interior vertical mortar | | | | | |
| (a) Stretcher, edge | 2¼" | .2621 | 2.743 | 1.391 | 3.816 | 1.521 | .0100 |
| (b) Stretcher, flat | 3¾" | .1722 | 4.174 | 1.391 | 5.807 | 1.000 | .0147 |
| (c) Header, flat | 8" | .0873 | 4.174 | 2.742 | 11.448 | 1.000 | .0192 |
| (d) Rowlock, edge | 8" | .0873 | 2.743 | 4.173 | 11.448 | 1.521 | .0192 |
| (e) Soldier, end | 3¾" | .1722 | 1.393 | 4.173 | 5.807 | 3.000 | .0147 |

Add interior vertical mortar joint between wythes — ½" thick per sq. ft. of wall = .0417 cu. ft. of mortar per sq. ft.

## ESTIMATING TABLES FOR CLAY PRODUCTS MASONRY

Complete tables giving quantities for all types of masonry walls now used commercially would be quite voluminous; however, it is believed that the following condensed tables can be used as a basis for estimating the material requirements for practically all types of clay products masonry.

Some variation will be found in all tables of mortar quantities. This is due to the weights per cu. ft. which have been used as a basis. Cement, lime and sand may vary some from the standard weights used. The more accurate method of proportioning mortar is by weight. Proportions are given in the following tables by both weight and volume.

The degree of dampness and compaction of sand causes a wide range in weights per cu. ft. In these tables, the volume of loose, damp sand is considered, while the weight is that of the dry sand normally contained in that volume of loose, damp sand. Sand is generally sold by the ton, and in its usual damp condition weighs approximately 100 lb. per cu. ft. In other words, a ton of damp sand normally contains 20 cu. ft. as given in the volume proportions, but only 1600 lb. of dry sand as listed in weight proportions.

For a large quantity of mortar, it is advisable to determine the weight per cu. ft. of the cement, lime and sand. Trial batches should be made and the yield accurately determined.

The number of masonry units given in Tables 62, 63 and 64 are net quantities. Some allowance should be made for breakage depending upon the design and construction conditions. As a general thing approximately 2% of the net quantity is added to provide this surplus for breakage.

## TABLE NO. 62

### SOLID BRICK WALLS IN RUNNING BOND
### (No headers, or pattern made with blind headers)

½" joints completely filled, including vertical interior joints of 8", 12" and 16" walls.

For estimating number of facing and backup brick in any particular type of bond, corrections should be made according to Table No. 2.

| Sq. Ft. Wall Area | 4" WALL | | 8" WALL | | 12" WALL | | 16" WALL | |
|---|---|---|---|---|---|---|---|---|
| | No. of Bricks | Cu. Ft. Mortar | No. of Bricks | Cu. Ft. Mortar | No. of Bricks | Cu. Ft. Mortar | No. of Bricks | Cu. Ft. Mortar |
| 1 | 6.16 | .0721 | 12.32 | .19 | 18.48 | .3 | 24.64 | .4132 |
| 10 | 62 | .721 | 124.0 | 1.86 | 185 | 3.0 | 247 | 4.2 |
| 20 | 124 | 1.5 | 247 | 3.72 | 370 | 6.0 | 493 | 8.3 |
| 30 | 185 | 2.2 | 370 | 5.6 | 555 | 9.0 | 740 | 12.4 |
| 40 | 247 | 2.9 | 493 | 7.5 | 740 | 12 | 986 | 16.6 |
| 50 | 308 | 3.6 | 616 | 9.3 | 924 | 15 | 1232 | 20.7 |
| 60 | 370 | 4.4 | 740 | 11.15 | 1109 | 18 | 1479 | 24.8 |
| 70 | 432 | 5.1 | 863 | 13 | 1294 | 21 | 1725 | 29 |
| 80 | 493 | 5.8 | 986 | 14.9 | 1479 | 24 | 1972 | 33.1 |
| 90 | 555 | 6.5 | 1109 | 16.75 | 1664 | 27 | 2218 | 37.2 |
| 100 | 616 | 7.25 | 1232 | 18.6 | 1848 | 30 | 2464 | 41.4 |
| 200 | 1232 | 14.5 | 2464 | 37.2 | 3696 | 60 | 4928 | 82.7 |
| 300 | 1848 | 21.7 | 3696 | 55.75 | 5544 | 90 | 7392 | 124 |
| 400 | 2464 | 29 | 4928 | 74.33 | 7392 | 120 | 9856 | 165.3 |
| 500 | 3080 | 36 | 6160 | 92.90 | 9240 | 150 | 12320 | 207 |
| 600 | 3696 | 43.3 | 7392 | 111.5 | 11088 | 180 | 14784 | 248 |
| 700 | 4312 | 50.5 | 8624 | 130.1 | 12936 | 210 | 17248 | 289.3 |
| 800 | 4928 | 57.7 | 9856 | 148.7 | 14784 | 240 | 19712 | 330.6 |
| 900 | 5544 | 64.9 | 11088 | 167.25 | 16632 | 270 | 22176 | 372 |
| 1000 | 6160 | 72.1 | 12320 | 185.8 | 18480 | 300 | 24640 | 413.2 |

## TABLE NO. 63

### CORRECTION FACTORS TO BE APPLIED TO 4″ WALL QUANTITIES FROM TABLE NO. 1 FOR ESTIMATING THE REQUIRED NUMBER OF FACING BRICK IN VARIOUS BONDS

**(Add to facing brick and deduct from backup)**

| TYPE OF BOND | CORRECTION FACTOR |
|---|---|
| Common bond with full header every 5th course | 20% or 1/5 |
| Common bond with full header every 6th course | 16.7% or 1/6 |
| Common bond with full header every 7th course | 14.3% or 1/7 |
| English bond, alternate courses full headers | 50% or 1/2 |
| English or Dutch bond with full headers every 6th course and blind headers in intermediate courses | 16.7% or 1/6 |
| Flemish bond, alternate stretchers and full headers every course | 33.3% or 1/3 |
| Flemish bond, with stretchers and full headers every 6th course, intermediate courses with blind headers | 5.6% or 1/18 |
| Flemish cross bond, alternate courses with stretchers and headers | 16.7% or 1/6 |
| Double header and stretcher every 6th course | 8.3% or 1/12 |
| Double header and stretcher every 5th course | 10% or 1/10 |

## TABLE NO. 64

### HOLLOW TILE WALLS

**(Side Construction)**

½″ joints; full bed joint with ends of inner and outer vertical shells buttered. (No mortar in interior joints parallel to face of wall).

| SQUARE FEET WALL AREA | 4″ WALLS | | 8″ WALLS | |
|---|---|---|---|---|
| | No. of 3¾x5x12 | Cu. Ft. Mortar | No. of 8x5x12 | Cu. Ft. Mortar |
| 1 | 2.1 | .034 | 2.1 | .065 |
| 10 | 21 | .34 | 21 | .65 |
| 20 | 42 | .68 | 42 | 1.3 |
| 30 | 63 | 1.02 | 63 | 1.94 |
| 40 | 84 | 1.4 | 84 | 2.6 |
| 50 | 105 | 1.7 | 105 | 3.25 |
| 60 | 126 | 2.1 | 126 | 3.9 |
| 70 | 147 | 2.4 | 147 | 4.75 |
| 80 | 168 | 2.75 | 168 | 5.2 |
| 90 | 189 | 3.1 | 189 | 5.9 |
| 100 | 209 | 3.4 | 209 | 6.5 |
| 200 | 418 | 6.8 | 418 | 13 |
| 300 | 627 | 10.2 | 627 | 19.4 |
| 400 | 836 | 13.6 | 836 | 26 |
| 500 | 1045 | 17 | 1045 | 32.4 |
| 600 | 1254 | 20.4 | 1254 | 38.8 |
| 700 | 1463 | 23.8 | 1463 | 47.25 |
| 800 | 1672 | 27.2 | 1672 | 51.7 |
| 900 | 1881 | 30.6 | 1881 | 58.2 |
| 1000 | 2090 | 34.0 | 2090 | 64.7 |

## TABLE NO. 64 (Continued)

| SQ. FT. WALL AREA | 12" WALL BONDED EA. COURSE | | | 16" WALL BONDED EA. COURSE | | |
|---|---|---|---|---|---|---|
| | No. of 3¾x5x12 | No. of 8x5x12 | Cu. Ft. Mortar | No. of 3¾x5x12 | No. of 8x5x12 | Cu. Ft. Mortar |
| 1 | 2.1 | 2.1 | .1 | 2.1 | 3.14 | .131 |
| 10 | 21 | 21 | 1 | 21 | 32 | 1.31 |
| 20 | 42 | 42 | 2 | 42 | 63 | 2.6 |
| 30 | 63 | 63 | 3 | 63 | 95 | 4 |
| 40 | 84 | 84 | 4 | 84 | 126 | 6 |
| 50 | 105 | 105 | 5 | 105 | 157 | 7 |
| 60 | 126 | 126 | 6 | 126 | 189 | 8 |
| 70 | 147 | 147 | 7 | 147 | 220 | 10 |
| 80 | 168 | 168 | 8 | 168 | 252 | 11 |
| 90 | 189 | 189 | 9 | 189 | 283 | 12 |
| 100 | 209 | 209 | 10 | 209 | 314 | 14 |
| 200 | 418 | 418 | 20 | 418 | 628 | 27 |
| 300 | 627 | 627 | 30 | 627 | 942 | 40 |
| 400 | 836 | 836 | 40 | 836 | 1256 | 53 |
| 500 | 1045 | 1045 | 50 | 1045 | 1570 | 66 |
| 600 | 1254 | 1254 | 60 | 1254 | 1884 | 80 |
| 700 | 1463 | 1463 | 70 | 1463 | 2198 | 92 |
| 800 | 1672 | 1672 | 80 | 1672 | 2512 | 105 |
| 900 | 1881 | 1881 | 90 | 1881 | 2826 | 118 |
| 1000 | 2090 | 2090 | 100 | 2090 | 3140 | 131 |

In estimating quantities in a Brick and Tile Wall, the usual procedure is to estimate the number of facing and backup brick required, and then substitute hollow tile for the backup brick according to the following table of equivalents.

## TABLE NO. 65

### BRICK AND TILE EQUIVALENTS

**Mortar required for 1000 tile in ½" joints (none on inside face)**

| SIZE OF TILE | POSITION LAID | EQUIVALENT BRICK UNITS | CU. FT. MORTAR PER 1000 TILE |
|---|---|---|---|
| 3¾x5x12 | Side | 2.91 | 16 |
| 8x5x12 | Side | 5.82 | 31 |
| 8x8x8 cube | Side | 6.2 | 25 |
| 8x12x12 | Side | 11.34 | 35 |
| 8x12x12 | End | 11.34 | 31 |
| 3¾x12x12 | Side | 5.67 | 20 |
| 3¾x12x12 | End | 5.67 | 20 |
| 12x12x12 | Side | 17.01 | 50 |
| 12x12x12 | End | 17.01 | 35 |

For 12" and 16" walls, the estimator should keep in mind the necessity for backup brick to complete the course behind the facing brick headers. The number of such backup brick required should be deducted from the total of backup brick to be replaced with tile.

**TABLE NO. 66**

**Mortar Required for 1000 Building Brick Laid in ½" Joints (none on inside face)**

| POSITION LAID | CU. FT. OF MORTAR |
|---|---|
| Stretcher (flat) | 11.7 |
| Stretcher (edge) | 7.9 |
| Header | 14.9 |
| Rowlock | 14.9 |
| Soldier | 11.7 |

**TABLE NO. 67**

**Mortar Required for 1000 Glazed Brick (none on inside face)**

| SIZE OF FACE | BLDG. BRICK EQUIV-ALENT | CU. FT. MORTAR REQUIRED | |
|---|---|---|---|
| | | ¼" JOINTS | ⅜" JOINTS |
| 2¼"x8" | 1 | 6.1 | 9.2 |
| 5"x8" | 2 | 7.7 | 11.6 |
| 5"x12" | 3 | 10 | 15 |

Mortar for Interior Vertical Joints (parging or plastering) amounts to:
3.13 cu. ft. for 100 sq. ft. of mortar joint ⅜" thick, and
4.17 cu. ft. for 100 sq. ft. of mortar joint ½" thick.

The following mixes are classified according to the volume proportions of Portland cement, hydrated lime and sand. Cement is assumed as one bag equal to one cu. ft. and weighing 94 lbs. Hydrated lime is assumed at 40 lbs. per cu. ft., and 50 lbs. to the bag. Sand has been based on average conditions, in which a cubic foot of loose, damp sand, as normally used in construction, contains approximately 80 lbs. of dry sand.

**TABLE NO. 68**

**QUANTITIES OF MORTAR MATERIALS**

| Cu. Ft. of Mortar | 1-0.25-3 MIX | | | | | | 1-1-6 MIX | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | WEIGHT | | | VOLUME | | | WEIGHT | | | VOLUME | | |
| | Cem. | Hyd. Lime | Sand | Cem. | Hyd. Lime | Sand | Cem. | Hyd. Lime | Sand | Cem. | Hyd. Lime | Sand |
| | lb. | lb. | lb. | Sack | Sack | Cu.Ft. | lb. | lb. | lb. | Sack | Sack | Cu.Ft. |
| 1 | 28 | 3 | 74 | 0.3 | 0.06 | 0.92 | 15.3 | 6.5 | 78.4 | 0.16 | 0.13 | 1 |
| 2 | 56 | 6 | 147 | 0.6 | 0.12 | 1.8 | 31 | 13 | 157 | 0.3 | 0.3 | 2 |
| 3 | 85 | 9 | 221 | 0.9 | 0.18 | 2.8 | 46 | 20 | 235 | 0.5 | 0.4 | 3 |
| 4 | 113 | 12 | 294 | 1.2 | 0.24 | 3.7 | 61 | 26 | 314 | 0.7 | 0.5 | 3.9 |
| 5 | 141 | 15 | 368 | 1.5 | 0.30 | 4.6 | 76 | 33 | 392 | 0.8 | 0.7 | 4.9 |
| 6 | 169 | 18 | 442 | 1.8 | 0.36 | 5.5 | 92 | 39 | 470 | 1.0 | 0.8 | 5.9 |
| 7 | 197 | 21 | 515 | 2.1 | 0.42 | 6.4 | 107 | 46 | 549 | 1.1 | 0.9 | 6.9 |
| 8 | 226 | 24 | 589 | 2.4 | 0.48 | 7.4 | 122 | 52 | 627 | 1.3 | 1.0 | 7.9 |
| 9 | 254 | 27 | 662 | 2.7 | 0.54 | 8.3 | 138 | 59 | 706 | 1.5 | 1.2 | 8.8 |
| 10 | 282 | 30 | 736 | 3.0 | 0.6 | 9.2 | 153 | 65 | 784 | 1.63 | 1.3 | 9.8 |
| 20 | 564 | 60 | 1472 | 6.0 | 1.2 | 18.4 | 306 | 130 | 1568 | 3.3 | 2.6 | 19.7 |
| 27 | 761 | 81 | 1987 | 8.1 | 1.6 | 24.8 | 413 | 176 | 2117 | 4.4 | 3.5 | 26.5 |
| 30 | 846 | 90 | 2208 | 9.0 | 1.8 | 27.6 | 459 | 195 | 2352 | 4.9 | 3.9 | 29.5 |
| 40 | 1128 | 120 | 2944 | 12 | 2.4 | 36.8 | 611 | 260 | 3136 | 6.5 | 5.2 | 39.3 |
| 50 | 1410 | 150 | 3680 | 15 | 3.0 | 46 | 764 | 325 | 3920 | 8.2 | 6.5 | 49.2 |
| 60 | 1692 | 180 | 4416 | 18 | 3.6 | 55 | 917 | 390 | 4704 | 9.8 | 7.8 | 59.0 |
| 70 | 1974 | 210 | 5152 | 21 | 4.2 | 64 | 1070 | 455 | 5488 | 11.4 | 9.1 | 68.8 |
| 80 | 2256 | 240 | 5888 | 24 | 4.8 | 74 | 1223 | 520 | 6272 | 13.0 | 10.4 | 78.6 |
| 90 | 2538 | 270 | 6624 | 27 | 5.4 | 83 | 1375 | 585 | 7056 | 14.7 | 11.7 | 88.5 |
| 100 | 2820 | 300 | 7360 | 30 | 6.0 | 92 | 1528 | 650 | 7840 | 16.3 | 13 | 98.3 |

## TABLE NO. 68—Continued

| CU. FT. OF MORTAR | 1-2-9 MIX | | | | | |
|---|---|---|---|---|---|---|
| | WEIGHT | | | VOLUME | | |
| | Cem. | Hyd. Lime | Sand | Cem. | Hyd. Lime | Sand |
| | lb. | lb. | lb. | Sack | Sack | Cu. Ft. |
| 1 | 10.3 | 8.75 | 79.2 | 0.11 | 0.18 | 1 |
| 2 | 21 | 18 | 158 | 0.2 | 0.4 | 2 |
| 3 | 31 | 26 | 238 | 0.3 | 0.5 | 3 |
| 4 | 41 | 35 | 317 | 0.4 | 0.7 | 4 |
| 5 | 52 | 44 | 396 | 0.6 | 0.9 | 5 |
| 6 | 62 | 53 | 475 | 0.7 | 1.1 | 5.9 |
| 7 | 72 | 61 | 554 | 0.8 | 1.2 | 6.9 |
| 8 | 83 | 70 | 634 | 0.9 | 1.4 | 7.9 |
| 9 | 93 | 79 | 713 | 1.0 | 1.6 | 8.9 |
| 10 | 103 | 87.5 | 792 | 1.1 | 1.75 | 9.9 |
| 20 | 207 | 175 | 1584 | 2.2 | 3.5 | 19.8 |
| 27 | 279 | 236 | 2138 | 3.0 | 4.7 | 26.7 |
| 30 | 310 | 263 | 2376 | 3.3 | 5.3 | 29.7 |
| 40 | 414 | 350 | 3168 | 4.4 | 7.0 | 39.6 |
| 50 | 517 | 438 | 3960 | 5.5 | 8.8 | 49.5 |
| 60 | 620 | 525 | 4752 | 6.6 | 10.5 | 59.4 |
| 70 | 724 | 613 | 5544 | 7.7 | 12.3 | 69.3 |
| 80 | 827 | 700 | 6336 | 8.8 | 14.0 | 79.2 |
| 90 | 931 | 788 | 7128 | 9.9 | 15.8 | 89.1 |
| 100 | 1034 | 875 | 7920 | 11 | 17.5 | 99 |

### EXAMPLES

(Problem No. 1) Estimate brick and mortar required in 800 sq. ft. of 12″ wall with ½″ joints in a 1-0.25-3 mortar.

(Solution) It is assumed that all brick are the same quality, therefore, from Table No. 62, it is noted that 14,784 brick will be required, and 240 cu. ft. mortar; adding approximately 2% of brick for breakage, the total brick to be purchased is 15,100. From Table No. 68, we find that 240 cu. ft. of 1-0.25-3 mortar requires 72 sacks cement, 14.4 sacks hydrated lime and 220.8 cu. ft. sand (approximately 11 tons).

(Problem No. 2) In the above problem, estimate the quantity of facing and backup brick required, wall in common running bond with headers every 7th course; ½″ joints in 1-1-6 mortar.

(Solution)

**Facing Brick**

From Table No. 62, 800 sq. ft. of 4″ wall requires.....4928
From Table No. 63, headers every 7th course, add 1/7 of above 704

Net quantity of facing brick required.....5632
Adding approximately 2% for breakage..... 118

Total number of facing brick.....5750

**Backup Brick**

| | |
|---|---|
| From Table No. 62, 800 sq. ft. of 8″ wall requires | 9856 |
| Deducting for facing headers above | 704 |
| Net number of backup brick required | 9152 |
| Adding approximately 2% for breakage | 198 |
| Total number of backup brick | 9350 |

Mortar required is same as in Problem No. 1, namely, 240 cu. ft. The mix is different (1-1-6) and from Table No. 68, we find that 240 cu.ft. of 1-1-6 mortar requires 39.1 sacks cement, 31.2 sacks hydrated lime and 235.9 cu. ft. of sand (approximately 11.8 tons).
(Problem No. 3) Same as Problem No. 2, except that hollow tile is used for backup.

(Solution)

**Facing Brick**

Same as Problem No. 2, total required 5750.

**Hollow Tile**

Backup brick required in Problem No. 2, was 9152. However, some backup brick will be required to finish the course behind the headers. In this problem (12″ wall), such a backup course is 4″ wide in every 7th course. Therefore, the number of brick required for backup is 1/7 of 4928, or 704.

Deducting this amount from the original net total of backup brick (9152—704= 8448), gives us the number to be replaced by hollow tile. We shall use 8x5x12 tile for the 8″ backing and from Table No. 65, we learn that one tile 8x5x12 is equivalent to 5.82 brick. Therefore, the net number of 8x5x12 tile is 8448/5.82 = 1452, and adding approximately 2% for breakage, the total required is 1500.

**Backup Brick**

In determining the amount of backup brick to be replaced by hollow tile, it was found that number of backup brick still required to complete header courses was 704 net, and with approximately 2% added, the total required is 720.

**Mortar**

In walls of various combinations of tile and brick, it is preferable to determine the net number of each unit required and then obtain the amount of mortar required for each group of units. In this wall, we will use 4928 building brick as stretchers, 1408 brick as headers and 1452 of 8x5x12 tile. From Tables No. 65 and No. 66, we obtain mortar quantities for tile and brick units, respectively.

| | | | |
|---|---|---|---|
| 4928 stretcher brick require | 11.7x4.928 | = | 57.6 |
| 1408 header brick require | 14.9x1.408 | = | 21.0 |
| 1452 tile (8x5x12) require | .31x1.452 | = | 45.0 |
| *800 sq. ft. of interior vertical joint | 4.17x8.00 | = | 33.4 |
| Total mortar required | | | 157.0 Cu. Ft. |

* Mortar quantities per unit of brick or tile do not include mortar on the interior face of the unit. If the interior vertical joints are filled with mortar, this item should be added. Quantities for 100 sq. ft. of ⅜″ and ½″ mortar joints or parging are given at the foot of Tables 5 and 6.

## TABLE 69

## QUANTITY OF MATERIALS REQUIRED FOR MASONRY MORTAR

### Lump Quicklime

**(70 Cu. Ft. Lime Putty per Ton of Lump Quicklime)**

| Proportions by Volume | | | QUANTITY OF MATERIALS REQUIRED | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | For one cubic yard of mortar | | | To lay 1000 brick (17 cu. ft.) | | |
| Cement | Lime Putty | Sand | Cement Bags | Lime Pounds | Sand Cu. Yd. | Cement Bags | Lime Pounds | Sand Cu. Yd. |
| 0 | 1 | 3 | 0 | 257 | 1 | 0 | 162 | .63 |
| 1 | 3 | 12 | 2¼ | 193 | 1 | 1.42 | 121½ | .63 |
| 1 | 2 | 9 | 3 | 171½ | 1 | 1.89 | 108 | .63 |
| 1 | 1½ | 7½ | 3.6 | 154 | 1 | 2.27 | 97 | .63 |
| 1 | 1 | 6 | 4½ | 128½ | 1 | 2.83 | 81 | .63 |
| 1 | ½ | 4½ | 6 | 86 | 1 | 3.78 | 54 | .63 |
| 1 | 0 | 3 | 9 | 0 | 1 | 5.67 | 0 | .63 |
| 1 | *10% | 3 | 9 | 26 | 1 | 5.67 | 16 | .63 |
| 1 | *15% | 3 | 9 | 39 | 1 | 5.67 | 24 | .63 |

### Pulverized Quicklime

**(80 Cu. Ft. Lime Putty per Ton of Pulverized Quicklime)**

| Proportions by Volume | | | QUANTITY OF MATERIALS REQUIRED | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | For one cubic yard of mortar | | | To lay 1000 brick (17 cu. ft.) | | |
| Cement | Lime Putty | Sand | Cement Bags | Lime Pounds | Sand Cu. Yd. | Cement Bags | Lime Pounds | Sand Cu. Yd. |
| 0 | 1 | 3 | 0 | 225 | 1 | 0 | 142 | .63 |
| 1 | 3 | 12 | 2¼ | 169 | 1 | 1.42 | 106 | .63 |
| 1 | 2 | 9 | 3 | 150 | 1 | 1.89 | 94½ | .63 |
| 1 | 1½ | 7½ | 3.6 | 135 | 1 | 2.27 | 85 | .63 |
| 1 | 1 | 6 | 4½ | 112½ | 1 | 2.83 | 71 | .63 |
| 1 | ½ | 4½ | 6 | 75 | 1 | 3.78 | 47 | .63 |
| 1 | 0 | 3 | 9 | 0 | 1 | 5.67 | 0 | .63 |
| 1 | *10% | 3 | 9 | 22½ | 1 | 5.67 | 14 | .63 |
| 1 | *15% | 3 | 9 | 34 | 1 | 5.67 | 21 | .63 |

*Based on Volume of Cement Required.

No allowance for waste.

## TABLE NO. 69 (Continued)

## QUANTITY OF MATERIALS REQUIRED FOR MASONRY MORTAR—(Cont.)

### Hydrated Lime (46 Cu. Ft. Lime Putty per Ton of Hydrated Lime)

| Proportions by Volume | | | QUANTITY OF MATERIALS REQUIRED | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | For one cubic yard of mortar | | | To lay 1000 brick (17 cu. ft.) | | |
| Cement | Lime Putty | Sand | Cement Bags | Lime Pounds | Sand Cu. Yd. | Cement Bags | Lime Pounds | Sand Cu. Yd. |
| 0 | 1 | 3 | 0 | 391 | 1 | 0 | 246 | .63 |
| 1 | 3 | 12 | 2¼ | 293½ | 1 | 1.42 | 185 | .63 |
| 1 | 2 | 9 | 3 | 261 | 1 | 1.89 | 164 | .63 |
| 1 | 1½ | 7½ | 3.6 | 235 | 1 | 2.27 | 148 | .63 |
| 1 | 1 | 6 | 4½ | 195½ | 1 | 2.83 | 123 | .63 |
| 1 | ½ | 4½ | 6 | 130½ | 1 | 3.78 | 82 | .63 |
| 1 | 0 | 3 | 9 | 0 | 1 | 5.67 | 0 | .63 |
| 1 | *10% | 3 | 9 | 39 | 1 | 5.67 | 25 | .63 |
| 1 | *15% | 3 | 9 | 59 | 1 | 5.67 | 37 | .63 |

*Based on Volume of Cement Required.
No allowance for waste.

## TABLE NO. 70 — AREA OF CIRCLES

| Diameter (Inches) | AREA | | Diameter (Inches) | AREA | |
|---|---|---|---|---|---|
| | Square Inches | Square Feet | | Square Inches | Square Feet |
| ¼ | 0.0491 | 0.00034 | 30 | 706.9 | 4.90 |
| ½ | 0.1963 | 0.00136 | 32 | 804.3 | 5.58 |
| ¾ | 0.4417 | 0.00306 | 34 | 907.9 | 6.30 |
| 1 | 0.7854 | 0.00545 | 36 | 1018.0 | 7.07 |
| 1¼ | 1.227 | 0.00853 | 38 | 1134.0 | 7.88 |
| 1½ | 1.767 | 0.0123 | 40 | 1257.0 | 8.72 |
| 1¾ | 2.405 | 0.0167 | 42 | 1385.0 | 9.62 |
| 2 | 3.142 | 0.0218 | 44 | 1521.0 | 10.57 |
| 2½ | 4.909 | 0.0341 | 46 | 1662.0 | 11.53 |
| 3 | 7.069 | 0.0492 | 48 | 1810.0 | 12.56 |
| 3½ | 9.621 | 0.0668 | 50 | 1964.0 | 13.63 |
| 4 | 12.566 | 0.0872 | 52 | 2124.0 | 14.75 |
| 4½ | 15.909 | 0.1105 | 54 | 2290.0 | 15.90 |
| 5 | 19.635 | 0.1362 | 56 | 2463.0 | 17.10 |
| 6 | 28.27 | 0.196 | 58 | 2642.0 | 18.35 |
| 7 | 38.48 | 0.267 | 60 | 2827.0 | 19.62 |
| 8 | 50.26 | 0.349 | 62 | 3019.0 | 20.93 |
| 9 | 63.62 | 0.442 | 64 | 3217.0 | 22.3 |
| 10 | 78.54 | 0.545 | 66 | 3421.0 | 23.8 |
| 12 | 113.1 | 0.785 | 68 | 3632.0 | 25.2 |
| 14 | 153.9 | 1.068 | 70 | 3848.0 | 26.2 |
| 16 | 201.1 | 1.395 | 72 | 4072.0 | 28.3 |
| 18 | 254.5 | 1.765 | 76 | 4536.0 | 31.4 |
| 20 | 314.2 | 2.18 | 80 | 5027.0 | 34.9 |
| 22 | 380.1 | 2.64 | 90 | 6362.0 | 44.2 |
| 24 | 452.4 | 3.14 | 100 | 7854.0 | 54.5 |
| 26 | 530.9 | 3.68 | 110 | 9503.0 | 66.0 |
| 28 | 615.8 | 4.27 | 120 | 11310.0 | 78.5 |

## TABLE NO. 71

## HEIGHT OF SOLID AND IDEAL BRICKWORK BY COURSES

Based on Standard Brick 2¼″ x 3¾″ x 8″

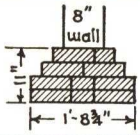
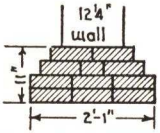
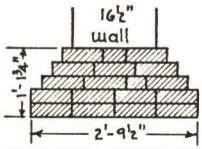
Height from Bottom of Mortar Joint to Bottom of Mortar Joint

| No. of Courses | ⅜″ Joints | | ½″ Joints | | ⅝″ Joints |
|---|---|---|---|---|---|
| | Brick Flat | Brick on Edge | Brick Flat | Brick on Edge | Brick Flat |
| 1 | 2 5/8″ | 4 1/8″ | 2 3/4″ | 4 1/4″ | 2 7/8″ |
| 2 | 5 1/4″ | 8 1/4″ | 5 1/2″ | 8 1/2″ | 5 3/4″ |
| 3 | 7 7/8″ | 1′— 0 3/8″ | 8 1/4″ | 1′— 0 3/4″ | 8 5/8″ |
| 4 | 10 1/2″ | 1′— 4 1/2″ | 11″ | 1′— 5″ | 11 1/2″ |
| 5 | 1′— 1 1/8″ | 1′— 8 3/8″ | 1′— 1 3/4″ | 1′— 9 1/4″ | 1′— 2 3/8″ |
| 6 | 1′— 3 3/4″ | 2′— 0 3/4″ | 1′— 4 1/2″ | 2′— 1 1/2″ | 1′— 5 1/4″ |
| 7 | 1′— 6 3/8″ | 2′— 4 7/8″ | 1′— 7 1/4″ | 2′— 5 3/4″ | 1′— 8 1/8″ |
| 8 | 1′— 9″ | 2′— 9″ | 1′—10″ | 2′—10″ | 1′—11″ |
| 9 | 1′—11 5/8″ | 3′— 1 1/8″ | 2′— 0 3/4″ | 3′— 2 1/4″ | 2′— 1 7/8″ |
| 10 | 2′— 2 1/4″ | 3′— 5 1/4″ | 2′— 3 1/2″ | 3′— 6 1/2″ | 2′— 4 3/4″ |
| 11 | 2′— 4 7/8″ | 3′— 9 3/8″ | 2′— 6 1/4″ | 3′—10 3/4″ | 2′— 7 5/8″ |
| 12 | 2′— 7 1/2″ | 4′— 1 1/2″ | 2′— 9″ | 4′— 3″ | 2′—10 1/2″ |
| 13 | 2′—10 1/8″ | 4′— 5 5/8″ | 2′—11 3/4″ | 4′— 7 1/4″ | 3′— 1 3/8″ |
| 14 | 3′— 0 3/4″ | 4′— 9 3/4″ | 3′— 2 1/2″ | 4′—11 1/2″ | 3′— 4 1/4″ |
| 15 | 3′— 3 3/8″ | 5′— 1 7/8″ | 3′— 5 1/4″ | 5′— 3 3/4″ | 3′— 7 1/8″ |
| 16 | 3′— 6″ | 5′— 6″ | 3′— 8″ | 5′— 8″ | 3′—10″ |
| 17 | 3′— 8 5/8″ | 5′—10 1/8″ | 3′—10 3/4″ | 6′— 0 1/4″ | 4′— 0 7/8″ |
| 18 | 3′—11 1/4″ | 6′— 2 1/4″ | 4′— 1 1/2″ | 6′— 4 1/2″ | 4′— 3 3/4″ |
| 19 | 4′— 1 7/8″ | 6′— 6 3/8″ | 4′— 4 1/4″ | 6′— 8 3/4″ | 4′— 6 5/8″ |
| 20 | 4′— 4 1/2″ | 6′—10 1/2″ | 4′— 7″ | 7′— 1″ | 4′— 9 1/2″ |
| 21 | 4′— 7 1/8″ | 7′— 2 5/8″ | 4′— 9 3/4″ | 7′— 5 1/4″ | 5′— 0 3/8″ |
| 22 | 4′— 9 3/4″ | 7′— 6 3/4″ | 5′— 0 1/2″ | 7′— 9 1/2″ | 5′— 3 1/4″ |
| 23 | 5′— 0 3/8″ | 7′—10 7/8″ | 5′— 3 1/4″ | 8′— 1 3/4″ | 5′— 6 1/8″ |
| 24 | 5′— 3″ | 8′— 3″ | 5′— 6″ | 8′— 6″ | 5′— 9″ |
| 25 | 5′— 5 5/8″ | 8′— 7 1/8″ | 5′— 8 3/4″ | 8′—10 1/4″ | 5′—11 7/8″ |
| 26 | 5′— 8 1/4″ | 8′—11 1/4″ | 5′—11 1/2″ | 9′— 2 1/2″ | 6′— 2 3/4″ |
| 27 | 5′—10 7/8″ | 9′— 3 3/8″ | 6′— 2 1/4″ | 9′— 6 3/4″ | 6′— 5 5/8″ |
| 28 | 6′— 1 1/2″ | 9′— 7 1/2″ | 6′— 5″ | 9′—11″ | 6′— 8 1/2″ |
| 29 | 6′— 4 1/8″ | 9′—11 5/8″ | 6′— 7 3/4″ | 10′— 3 1/4″ | 6′—11 3/8″ |
| 30 | 6′— 6 3/4″ | 10′— 3 3/4″ | 6′—10 1/2″ | 10′— 7 1/2″ | 7′— 2 1/4″ |
| 31 | 6′— 9 3/8″ | 10′— 7 7/8″ | 7′— 1 1/4″ | 10′—11 3/4″ | 7′— 5 1/8″ |
| 32 | 7′— 0″ | 11′— 0″ | 7′— 4″ | 11′— 4″ | 7′— 8″ |
| 33 | 7′— 2 5/8″ | 11′— 4 1/8″ | 7′— 6 3/4″ | 11′— 8 1/4″ | 7′—10 7/8″ |
| 34 | 7′— 5 1/4″ | 11′— 8 1/4″ | 7′— 9 1/2″ | 12′— 0 1/2″ | 8′— 1 3/4″ |
| 35 | 7′— 7 7/8″ | 12′— 0 3/8″ | 8′— 0 1/4″ | 12′— 4 3/4″ | 8′— 4 5/8″ |
| 36 | 7′—10 1/2″ | 12′— 4 1/2″ | 8′— 3″ | 12′— 9″ | 8′— 7 1/2″ |
| 37 | 8′— 1 1/8″ | 12′— 8 3/8″ | 8′— 5 3/4″ | 13′— 1 1/4″ | 8′—10 3/8″ |
| 38 | 8′— 3 3/4″ | 13′— 0 3/4″ | 8′— 8 1/2″ | 13′— 5 1/2″ | 9′— 1 1/4″ |
| 39 | 8′— 6 3/8″ | 13′— 4 7/8″ | 8′—11 1/4″ | 13′— 9 3/4″ | 9′— 4 1/8″ |
| 40 | 8′— 9″ | 13′— 9″ | 9′— 2″ | 14′— 2″ | 9′— 7″ |
| 41 | 8′—11 5/8″ | 14′— 1 1/8″ | 9′— 4 3/4″ | 14′— 6 1/4″ | 9′— 9 7/8″ |
| 42 | 9′— 2 1/4″ | 14′— 5 1/4″ | 9′— 7 1/2″ | 14′—10 1/2″ | 10′— 0 3/4″ |
| 43 | 9′— 4 7/8″ | 14′— 9 3/8″ | 9′—10 1/4″ | 15′— 2 3/4″ | 10′— 3 5/8″ |
| 44 | 9′— 7 1/2″ | 15′— 1 1/2″ | 10′— 1″ | 15′— 7″ | 10′— 6 1/2″ |
| 45 | 9′—10 1/8″ | 15′— 5 5/8″ | 10′— 3 3/4″ | 15′—11 1/4″ | 10′— 9 3/8″ |
| 46 | 10′— 0 3/4″ | 15′— 9 3/4″ | 10′— 6 1/2″ | 16′— 3 1/2″ | 11′— 0 1/4″ |
| 47 | 10′— 3 3/8″ | 16′— 1 7/8″ | 10′— 9 1/4″ | 16′— 7 3/4″ | 11′— 3 1/8″ |
| 48 | 10′— 6″ | 16′— 6″ | 11′— 0″ | 17′— 0″ | 11′— 6″ |
| 49 | 10′— 8 5/8″ | 16′—10 1/8″ | 11′— 2 3/4″ | 17′— 4 1/4″ | 11′— 8 7/8″ |
| 50 | 10′—11 1/4″ | 17′— 2 1/4″ | 11′— 5 1/2″ | 17′— 8 1/2″ | 11′—11 3/4″ |
| 60 | 13′— 1 1/2″ | 20′— 7 1/2″ | 13′—10″ | 21′— 3″ | 14′— 4 1/2″ |
| 70 | 15′— 3 3/4″ | 24′— 0 3/4″ | 16′— 0 1/2″ | 24′— 9 1/2″ | 16′— 9 1/4″ |
| 80 | 17′— 6″ | 27′— 6″ | 18′— 4″ | 28′— 4″ | 19′— 2″ |
| 90 | 19′— 8 1/4″ | 30′—11 1/4″ | 20′— 7 1/2″ | 31′—10 1/2″ | 21′— 6 3/4″ |
| 100 | 21′—10 1/2″ | 34′— 4 1/2″ | 22′—11″ | 35′— 5″ | 23′—11 1/2″ |

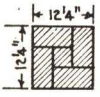
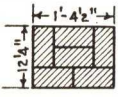
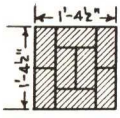
## TABLE NO. 72

## NUMBER OF BRICK AND QUANTITY OF MORTAR REQUIRED TO BUILD WALL FOOTINGS, PIERS AND CHIMNEYS

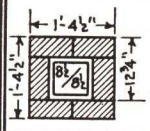
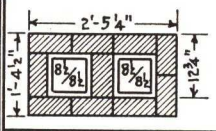
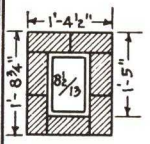
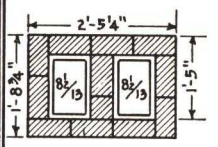
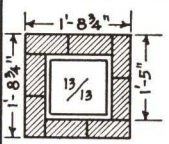
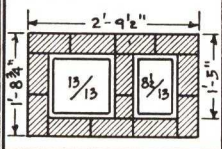
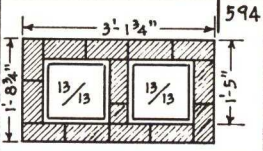
### FOOTINGS
Quantities for 100 lineal feet

| Section | No. of Brick | Cu. Ft. Mortr. |
|---|---|---|
|   | 2399 | 41.79 |
|   | 2964 | 52.38 |
|   | 4799 | 82.23 |

### PIERS
Quantities for 10 ft. ht. (44 courses)

| Plan | No. of Brick | Cu. Ft. Mortr. |
|---|---|---|
|   | 142 | 1.68 |
|   | 198 | 2.34 |
|   | 264 | 3.78 |
|   | 352 | 5.23 |
|   | 113 | 0.89 |

### CHIMNEYS
Quantities for 10 ft. ht. (44 courses)

| Plan | Free | | Attached | |
|---|---|---|---|---|
| | No. of Brick | Cu. Ft. Mortr. | No. of Brick | Cu. Ft. Mortr. |
|   | 264 | 4.76 | 176 | 3.73 |
|   | 440 | 8.49 | 286 | 6.69 |
|   | 308 | 5.79 | 220 | 4.76 |
|   | 506 | 10.30 | 352 | 8.50 |
|   | 352 | 6.81 | 242 | 5.52 |
|   | 550 | 11.31 | 374 | 9.26 |
|   | 594 | 12.34 | 396 | 10.02 |

## WEIGHT OF BRICK MASONRY

Because of the variation in the weight of brick units, the approximate weights of brick masonry have been estimated for different wall thicknesses. These weights are given in the following table for one square foot of wall area and are based on the brick weights indicated, plus the mortar required for ½″ joints completely filled; the mortar weight used being 116 pounds per cubic foot.

### TABLE NO. 73

### WEIGHT OF BRICK MASONRY WALLS IN POUNDS PER SQUARE FOOT FOR VARIOUS THICKNESSES OF WALLS

| Weight of Brick Unit | 2¼″ | 3¾″ | 8″ | 12¼″ | 16½″ |
|---|---|---|---|---|---|
| 4# brick..... | 19.53 | 31.79 | 68.41 | 105.03 | 141.65 |
| 4½# brick.... | 21.61 | 35.07 | 74.97 | 114.87 | 154.77 |
| 5# brick..... | 23.69 | 38.36 | 81.55 | 124.74 | 167.93 |
| 5½# brick.... | 25.77 | 41.64 | 88.11 | 134.58 | 181.05 |
| 6# brick..... | 27.85 | 44.92 | 94.77 | 144.52 | 194.27 |

Based on standard size brick, ½″ mortar joints filled, mortar at 116# per cu. ft.

## WEIGHT OF BRICK MASONRY BEAMS

Weights of brick masonry beams are given in the following tables. These are listed in pounds per lineal foot for the sizes noted and are estimated in two ways: (1) on the basis of the weight of the brick units and (2) the weights of a cubic foot of brick masonry.

### TABLE NO. 74

### WEIGHT OF BRICK MASONRY BEAMS IN POUNDS PER LINEAL FOOT

**(Based on various weights of brick unit)**

| Weight of brick | Width of Beam | Depth of Brick Masonry Beam | | | | |
|---|---|---|---|---|---|---|
| | | 13¾″ | 16½″ | 19¼″ | 22″ | 24¾″ |
| 4# brick.... | 8″ beam | 78.38 | 94.06 | 109.74 | 125.42 | 141.10 |
| | 12¼″ beam | 120.34 | 144.42 | 168.49 | 192.56 | 216.63 |
| 4½# brick.. | 8″ beam | 85.90 | 103.08 | 120.27 | 137.45 | 154.63 |
| | 12¼″ beam | 131.62 | 157.95 | 184.27 | 210.60 | 236.93 |
| 5# brick.... | 8″ beam | 93.44 | 112.13 | 130.82 | 149.51 | 168.20 |
| | 12¼″ beam | 142.92 | 171.52 | 200.11 | 228.70 | 257.29 |
| 5½# brick.. | 8″ beam | 100.96 | 121.15 | 141.35 | 161.54 | 181.74 |
| | 12¼″ beam | 154.20 | 185.05 | 215.89 | 246.74 | 277.58 |
| 6# brick.... | 8″ beam | 108.59 | 130.31 | 152.03 | 173.75 | 195.47 |
| | 12¼″ beam | 165.59 | 198.62 | 231.74 | 264.86 | 297.98 |

Based on standard size brick, ½″ mortar joints filled, mortar at 116# per cu. ft.

## TABLE NO. 75

## WEIGHTS OF REINFORCED BRICK MASONRY BEAMS IN POUNDS PER LINEAR FOOT

**(Based on various weights of a cubic foot of masonry)**

| Beam Size (Inches) | Weight of Brick Masonry per cubic foot | | | | | | |
|---|---|---|---|---|---|---|---|
| | 110 | 115 | 120 | 125 | 130 | 135 | 140 |
| 8x13¾.... | 84.0 | 87.8 | 91.7 | 95.5 | 99.3 | 103.2 | 107.1 |
| 8x16½.... | 100.8 | 105.4 | 110.0 | 114.6 | 119.2 | 123.8 | 128.3 |
| 8x19¼.... | 117.7 | 123.0 | 128.4 | 133.7 | 139.0 | 144.4 | 149.7 |
| 12x19¼... | 176.5 | 184.5 | 192.6 | 200.6 | 208.6 | 216.7 | 224.7 |
| 12x22..... | 201.7 | 210.8 | 220.0 | 229.2 | 238.3 | 247.5 | 256.7 |
| 12x24¾... | 227.1 | 237.4 | 247.7 | 258.0 | 268.3 | 278.6 | 289.0 |

## TABLE NO. 76

## AREAS, PERIMETERS AND WEIGHTS OF BARS

| Number of Bars | | Size of Bars | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | ¼-In. Round | ⅜-In. Round | ½-In. Round | ½-In. Round | ⅝-In. Round | ¾-In. Round | ⅞-In. Round | 1-In. Round |
| 1..... | A | 0.0491 | 0.1104 | 0.1963 | 0.250 | 0.3068 | 0.4418 | 0.6013 | 0.7854 |
| | O | 0.785 | 1.178 | 1.571 | 2.000 | 1.964 | 2.356 | 2.749 | 3.142 |
| | W | 0.167 | 0.375 | 0.668 | 0.850 | 1.043 | 1.502 | 2.044 | 2.670 |
| 2.....Sum. | A | 0.10 | 0.22 | 0.39 | 0.50 | 0.61 | 0.88 | 1.20 | 1.57 |
| | O | 1.57 | 2.36 | 3.14 | 4.00 | 3.93 | 4.71 | 5.50 | 6.28 |
| | W | 0.33 | 0.75 | 1.34 | 1.70 | 2.09 | 3.00 | 4.09 | 5.34 |
| 3.....Sum. | A | 0.15 | 0.33 | 0.59 | 0.75 | 0.92 | 1.33 | 1.80 | 2.36 |
| | O | 2.36 | 3.53 | 4.71 | 6.00 | 5.89 | 7.07 | 8.25 | 9.42 |
| | W | 0.50 | 1.12 | 2.00 | 2.55 | 3.13 | 4.51 | 6.13 | 8.01 |
| 4.....Sum. | A | 0.20 | 0.44 | 0.78 | 1.00 | 1.23 | 1.77 | 2.41 | 3.14 |
| | O | 3.14 | 4.71 | 6.28 | 8.00 | 7.86 | 9.42 | 11.00 | 12.57 |
| | W | 0.67 | 1.50 | 2.67 | 3.40 | 4.17 | 6.01 | 8.18 | 10.68 |
| Area of 12 Bars. | | 0.59 | 1.32 | 2.36 | 3.00 | 3.68 | 5.30 | 7.22 | 9.41 |

A = area in square inches.
O = perimeter in square inches per inch of length.
W = weight per linear foot.

## TABLE NO. 77

## REINFORCING BAR SIZES AND EQUIVALENT AREAS

| Area of rods in sq. in. | Number and Size of Rods | | | | Area of rods in sq. in. | Number and Size of Rods | | | |
|---|---|---|---|---|---|---|---|---|---|
| | ¼"Ø | ⅜"Ø | ½"Ø | ½"□ | | ¼"Ø | ⅜"Ø | ½"Ø | ½"□ |
| 0.098 | 2 | | | | 0.834 | 17 | | | |
| 0.147 | 3 | | | | 0.884 | 18 | 8 | | |
| 0.196 | 4 | | 1 | | 0.933 | 19 | | | |
| 0.221 | | 2 | | | 0.982 | 20 | | 5 | |
| 0.246 | 5 | | | | 0.994 | | 9 | | |
| 0.250 | | | | 1 | 1.00 | | | | 4 |
| 0.295 | 6 | | | | 1.08 | 22 | | | |
| 0.331 | | 3 | | | 1.104 | | 10 | | |
| 0.344 | 7 | | | | 1.178 | 24 | | 6 | |
| 0.393 | 8 | | 2 | | 1.214 | | 11 | | |
| 0.442 | 9 | 4 | | | 1.25 | | | | 5 |
| 0.491 | 10 | | | | 1.325 | | 12 | | |
| 0.500 | | | | 2 | 1.374 | | | 7 | |
| 0.540 | 11 | | | | 1.436 | | 13 | | |
| 0.552 | | 5 | | | 1.50 | | | | 6 |
| 0.589 | 12 | | 3 | | 1.546 | | 14 | | |
| 0.638 | 13 | | | | 1.57 | | | 8 | |
| 0.663 | | 6 | | | 1.656 | | 15 | | |
| 0.686 | 14 | | | | 1.75 | | | | 7 |
| 0.736 | 15 | | | | 1.767 | | 16 | 9 | |
| 0.750 | | | | 3 | 1.877 | | 17 | | |
| 0.773 | | 7 | | | 1.963 | | | 10 | |
| 0.785 | 16 | | 4 | | 2.000 | | | | 8 |

The number of bars listed in any column have the total cross-sectional area indicated in the column on left.

## TABLE NO. 78

**Values of p, k, j and K for Various Combinations of Steel and Brick Stresses**

For Rectangular Beams and Slabs

$$k = \frac{1}{1+\frac{f_s}{nf_b}} \qquad p = \frac{f_b k}{2f_s} \qquad j = 1 - \frac{k}{3} \qquad K = \frac{1}{2} f_b k j \text{ or } p f_s j.$$

**$n$ = 25**

**$f_s$ = 16,000**

| $f_b$ | 400 | 450 | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 900 | 1000 | 1100 | 1200 | 1300 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | .0048 | .0058 | .0069 | .0079 | .0091 | .0102 | .0114 | .0126 | .0139 | .0164 | .0191 | .0217 | .0245 | .0272 |
| $k$ | .3846 | .4128 | .4386 | .4622 | .4839 | .5039 | .5224 | .5396 | .5556 | .5844 | .6098 | .6322 | .6522 | .6701 |
| $j$ | .8718 | .8624 | .8538 | .8459 | .8387 | .8320 | .8259 | .8201 | .8148 | .8052 | .7967 | .7893 | .7826 | .7766 |
| $K$ | 67.06 | 80.10 | 93.62 | 107.52 | 121.75 | 136.26 | 151.01 | 165.95 | 181.06 | 211.75 | 242.91 | 274.45 | 306.25 | 338.26 |

**$f_s$ = 18,000**

| $f_b$ | 400 | 450 | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 900 | 1000 | 1100 | 1200 | 1300 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | .0040 | .0048 | .0057 | .0066 | .0076 | .0086 | .0096 | .0106 | .0117 | .0139 | .0161 | .0184 | .0208 | .0232 |
| $k$ | .3571 | .3846 | .4098 | .4331 | .4545 | .4745 | .4930 | .5102 | .5263 | .5556 | .5814 | .6044 | .6250 | .6436 |
| $j$ | .8810 | .8718 | .8634 | .8556 | .8485 | .8418 | .8357 | .8300 | .8246 | .8148 | .8062 | .7985 | .7917 | .7855 |
| $K$ | 62.92 | 75.44 | 88.46 | 101.90 | 115.69 | 129.81 | 144.20 | 158.80 | 173.60 | 203.60 | 234.36 | 265.44 | 296.89 | 328.61 |

**$f_s$ = 20,000**

| $f_b$ | 400 | 450 | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 900 | 1000 | 1100 | 1200 | 1300 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | .0033 | .0040 | .0048 | .0056 | .0064 | .0073 | .0082 | .0091 | .0100 | .0119 | .0139 | .0159 | .0180 | .0215 |
| $k$ | .3333 | .360 | .3846 | .4074 | .4286 | .4483 | .4666 | .4838 | .5000 | .5294 | .5556 | .5789 | .6000 | .6599 |
| $j$ | .8889 | .880 | .8718 | .8642 | .8571 | .8506 | .8445 | .8387 | .8333 | .8235 | .8148 | .8070 | .8000 | .780 |
| $K$ | 59.26 | 71.28 | 83.82 | 96.82 | 110.21 | 123.93 | 137.92 | 152.17 | 166.67 | 196.18 | 203.60 | 256.95 | 288.0 | 334.62 |

## TABLE NO. 79

### Values of p, k, j and K for Various Combinations of Steel and Brick Stresses

For Rectangular Beams and Slabs

$n = 20$

$f_s = 16{,}000$

| $f_b$ | 450 | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 900 | 1000 | 1100 | 1200 | 1300 | 1400 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | .0051 | .0060 | .0070 | .0080 | .0091 | .0102 | .0113 | .0125 | .0149 | .0174 | .0199 | .0225 | .0251 | .0278 |
| $k$ | .360 | .3846 | .4074 | .4286 | .4483 | .4667 | .4839 | .500 | .5294 | .5556 | .5789 | .600 | .6190 | .6364 |
| $j$ | .880 | .8718 | .8642 | .8571 | .8506 | .8444 | .8387 | .8333 | .8235 | .8148 | .8070 | .800 | .7937 | .7879 |
| $K$ | 71.28 | 83.82 | 96.82 | 110.21 | 123.93 | 137.93 | 152.19 | 166.67 | 196.18 | 226.34 | 256.95 | 288.00 | 319.35 | 350.99 |

$f_s = 18{,}000$

| $f_b$ | 450 | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 900 | 1000 | 1100 | 1200 | 1300 | 1400 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | .0042 | .0050 | .0058 | .0067 | .0076 | .0085 | .0095 | .0105 | .0125 | .0146 | .0168 | .0190 | .0213 | .0237 |
| $k$ | .3333 | .3571 | .3794 | .4000 | .4193 | .4374 | .4545 | .4706 | .5000 | .5263 | .5500 | .5714 | .5910 | .6086 |
| $j$ | .8889 | .8810 | .8735 | .8667 | .8602 | .8542 | .8485 | .8431 | .8333 | .8246 | .8167 | .8095 | .8030 | .7971 |
| $K$ | 66.67 | 78.65 | 91.14 | 104.00 | 117.22 | 130.77 | 144.62 | 158.71 | 187.50 | 217.00 | 247.05 | 277.53 | 308.47 | 339.58 |

$f_s = 20{,}000$

| $f_b$ | 450 | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 900 | 1000 | 1100 | 1200 | 1300 | 1400 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | .0035 | .0042 | .0049 | .0056 | .0064 | .0072 | .0080 | .0089 | .0107 | .0125 | .0144 | .0164 | .0184 | .0204 |
| $k$ | .3103 | .3333 | .3548 | .3750 | .3939 | .4117 | .4286 | .4444 | .4737 | .5000 | .5238 | .5454 | .5652 | .5834 |
| $j$ | .8966 | .8899 | .8817 | .8750 | .8687 | .8628 | .8571 | .8519 | .8421 | .8333 | .8254 | .8182 | .8116 | .8055 |
| $K$ | 62.60 | 74.07 | 86.06 | 98.44 | 111.25 | 124.33 | 137.75 | 151.45 | 179.48 | 208.25 | 237.67 | 267.68 | 297.95 | 328.93 |

## TABLE NO. 80

### Values of p, k, j and K for Various Combinations of Steel and Brick Stresses

For Rectangular Beams and Slabs

$n = 18$

$f_s = 16{,}000$

| $f_b$ | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 900 | 1000 | 1100 | 1200 | 1300 | 1400 | 1500 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | .0056 | .0066 | .0076 | .0086 | .0096 | .0107 | .0118 | .01415 | .0165 | .0190 | .0215 | .0241 | .0268 | .0294 |
| $k$ | .360 | .3822 | .4030 | .4224 | .4405 | .4576 | .4737 | .5031 | .5294 | .5531 | .5745 | .5939 | .6117 | .6279 |
| $j$ | .880 | .8726 | .8657 | .8592 | .8532 | .8475 | .8421 | .8323 | .8235 | .8156 | .8085 | .8020 | .7961 | .7907 |
| $K$ | 79.20 | 91.72 | 104.66 | 117.95 | 131.55 | 145.43 | 159.56 | 186.44 | 217.98 | 248.11 | 278.69 | 309.60 | 340.88 | 372.36 |

$f_s = 18{,}000$

| $f_b$ | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 900 | 1000 | 1100 | 1200 | 1300 | 1400 | 1500 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | .0046 | .0054 | .0062 | .0071 | .0080 | .00893 | .0099 | .0118 | .0139 | .0169 | .0182 | .0204 | .0227 | .0250 |
| $k$ | .3333 | .3548 | .3750 | .3939 | .4117 | .4286 | .4444 | .4737 | .500 | .5238 | .5454 | .5652 | .5833 | .600 |
| $j$ | .889 | .882 | .875 | .869 | .863 | .8571 | .852 | .842 | .833 | .825 | .818 | .811 | .8056 | .800 |
| $K$ | 74.07 | 86.06 | 98.44 | 111.25 | 124.33 | 137.75 | 151.45 | 179.48 | 208.25 | 237.67 | 267.68 | 297.95 | 328.93 | 360.00 |

$f_s = 20{,}000$

| $f_b$ | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 900 | 1000 | 1100 | 1200 | 1300 | 1400 | 1500 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | .0039 | .0046 | .0053 | .0060 | .0068 | .0076 | .0084 | .0101 | .0118 | .0138 | .0156 | .0175 | .0195 | .0215 |
| $k$ | .3103 | .3311 | .3506 | .3691 | .3865 | .4030 | .4186 | .4475 | .4737 | .5000 | .5192 | .5392 | .5576 | .5745 |
| $j$ | .8966 | .8896 | .8831 | .8770 | .8712 | .8657 | .8605 | .8508 | .8421 | .8333 | .8269 | .8203 | .8141 | .8085 |
| $K$ | 69.55 | 81.00 | 92.88 | 105.20 | 117.86 | 130.82 | 144.08 | 171.33 | 199.45 | 229.17 | 257.60 | 287.50 | 317.76 | 348.37 |

## TABLE NO. 81

### Values of p, k, j and K for Various Combinations of Steel and Brick Stresses

For Rectangular Beams and Slabs

$n = 15$

$f_s = 14{,}000$

| $f_b$ | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 850 | 900 | 950 | 1000 | 1100 | 1200 | 1300 | 1400 | 1500 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | 0.0062 | 0.0073 | 0.0084 | 0.0095 | 0.0107 | 0.0119 | 0.0132 | 0.0145 | 0.0158 | 0.0171 | 0.0185 | 0.0213 | 0.0241 | 0.0270 | 0.0300 | 0.0330 |
| $k$ | 0.3488 | 0.3708 | 0.3913 | 0.4105 | 0.4286 | 0.4455 | 0.4615 | 0.4766 | 0.4909 | 0.5044 | 0.5173 | 0.5410 | 0.5625 | 0.5821 | 0.6000 | 0.6164 |
| $j$ | 0.8838 | 0.8764 | 0.8696 | 0.8632 | 0.8571 | 0.8515 | 0.8462 | 0.8411 | 0.8364 | 0.8319 | 0.8276 | 0.8197 | 0.8125 | 0.8060 | 0.8000 | 0.7945 |
| $K$ | 77.07 | 89.37 | 102.1 | 115.2 | 128.6 | 142.2 | 156.2 | 170.4 | 184.8 | 199.3 | 214.1 | 243.9 | 274.2 | 305.0 | 336.0 | 367.3 |

$f_s = 16{,}000$

| $f_b$ | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 850 | 900 | 950 | 1000 | 1100 | 1200 | 1300 | 1400 | 1500 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | 0.0050 | 0.0058 | 0.0068 | 0.0077 | 0.0087 | 0.0097 | 0.0107 | 0.0118 | 0.0129 | 0.0140 | 0.0151 | 0.0175 | 0.0199 | 0.0223 | 0.0248 | 0.0274 |
| $k$ | 0.3191 | 0.3402 | 0.3600 | 0.3786 | 0.3962 | 0.4128 | 0.4286 | 0.4435 | 0.4576 | 0.4711 | 0.4839 | 0.5077 | 0.5294 | 0.5493 | 0.5676 | 0.5844 |
| $j$ | 0.8936 | 0.8866 | 0.8800 | 0.8738 | 0.8679 | 0.8624 | 0.8571 | 0.8522 | 0.8475 | 0.8430 | 0.8387 | 0.8308 | 0.8235 | 0.8169 | 0.8108 | 0.8052 |
| $K$ | 71.29 | 82.95 | 95.04 | 107.5 | 120.4 | 133.5 | 146.9 | 160.6 | 174.5 | 188.6 | 202.9 | 232.0 | 261.6 | 291.7 | 322.1 | 352.9 |

$f_s = 18{,}000$

| $f_b$ | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 850 | 900 | 950 | 1000 | 1100 | 1200 | 1300 | 1400 | 1500 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | 0.0041 | 0.0048 | 0.0056 | 0.0063 | 0.0072 | 0.0080 | 0.0089 | 0.0098 | 0.0107 | 0.0117 | 0.0126 | 0.0146 | 0.0167 | 0.0188 | 0.0209 | 0.0232 |
| $k$ | 0.2941 | 0.3143 | 0.3333 | 0.3514 | 0.3684 | 0.3846 | 0.4000 | 0.4146 | 0.4288 | 0.4419 | 0.4545 | 0.4783 | 0.5000 | 0.5200 | 0.5385 | 0.5556 |
| $j$ | 0.9020 | 0.8952 | 0.8889 | 0.8829 | 0.8772 | 0.8718 | 0.8667 | 0.8618 | 0.8571 | 0.8527 | 0.8485 | 0.8406 | 0.8333 | 0.8267 | 0.8205 | 0.8148 |
| $K$ | 66.32 | 77.37 | 88.88 | 100.8 | 113.1 | 125.7 | 138.7 | 151.9 | 165.4 | 179.0 | 192.8 | 221.1 | 250.0 | 279.4 | 309.3 | 339.5 |

$f_s = 20{,}000$

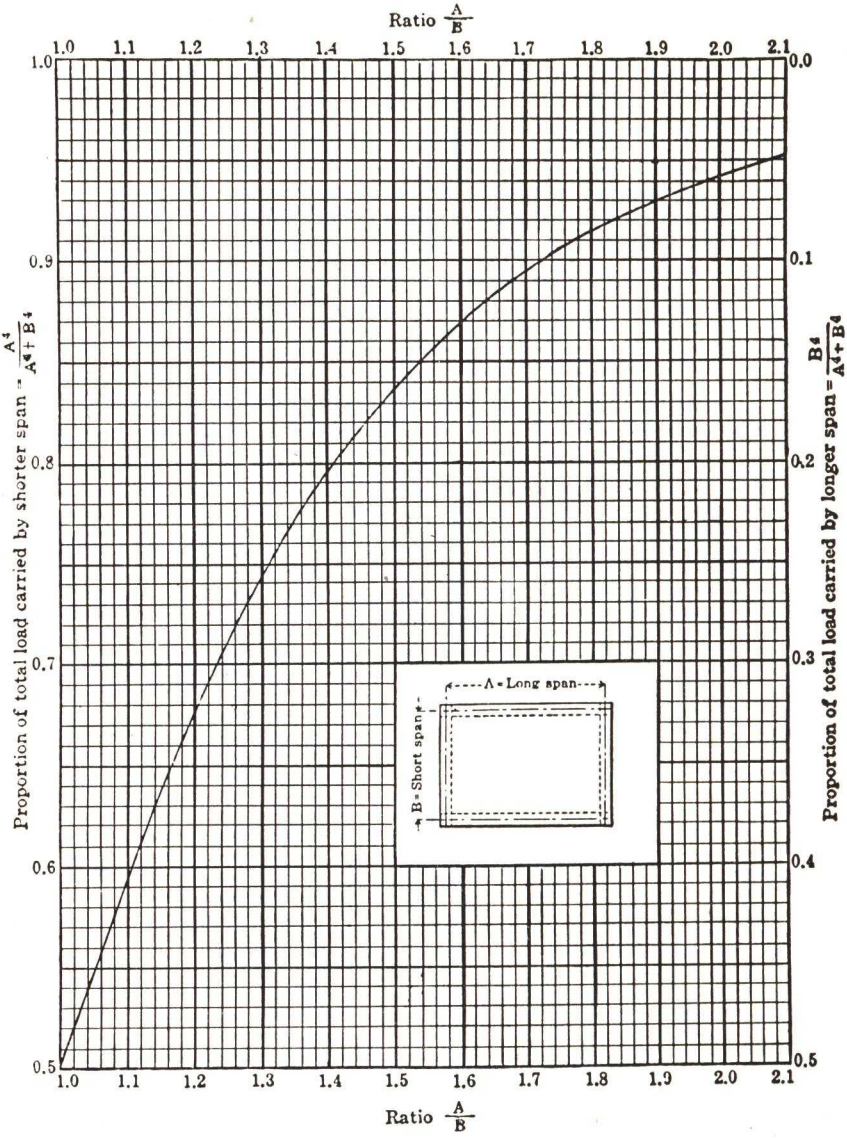
| $f_b$ | 500 | 550 | 600 | 650 | 700 | 750 | 800 | 850 | 900 | 950 | 1000 | 1100 | 1200 | 1300 | 1400 | 1500 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $p$ | 0.0034 | 0.0040 | 0.0047 | 0.0053 | 0.0060 | 0.0068 | **0.0075** | 0.0083 | **0.0091** | 0.0099 | 0.0107 | 0.0124 | 0.0142 | 0.0160 | 0.0179 | 0.0199 |
| $k$ | 0.2727 | 0.2920 | 0.3103 | 0.3277 | 0.3443 | 0.3600 | **0.3750** | 0.3893 | **0.4030** | 0.4161 | 0.4286 | 0.4521 | 0.4737 | 0.4937 | 0.5122 | 0.5294 |
| $j$ | 0.9091 | 0.9027 | 0.8966 | 0.8908 | 0.8852 | 0.8800 | **0.8750** | 0.8702 | **0.8657** | 0.8613 | 0.8571 | 0.8493 | 0.8421 | 0.8354 | 0.8293 | 0.8235 |
| $K$ | 61.98 | 72.49 | 83.46 | 94.87 | 106.7 | 118.8 | **131.3** | 144.0 | **157.0** | 170.2 | 183.7 | 211.2 | 239.3 | 268.1 | 297.3 | 327.0 |

## TABLE NO. 82

### Values of p, k, j and K for Various Combinations of Steel and Brick Stresses

For Rectangular Beams and Slabs

| | $n = 12$ | | | | | | | | | $n = 10$ | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $f_s = 16{,}000$ | | | | | | | | | $f_s = 16{,}000$ | | | | | | |
| $f_b$ | **800** | **900** | **1000** | **1100** | **1125** | **1200** | **1300** | **1400** | **1500** | **1000** | **1100** | **1200** | **1300** | **1350** | **1400** | **1500** |
| $p$ | 0.0094 | 0.0113 | 0.0134 | 0.0155 | 0.0161 | 0.0178 | 0.0201 | 0.0224 | 0.0248 | 0.0120 | 0.0140 | 0.0161 | 0.0182 | 0.0192 | 0.0204 | 0.0227 |
| $k$ | 0.3750 | 0.4030 | 0.4286 | 0.4521 | 0.4576 | 0.4737 | 0.4937 | 0.5122 | 0.5294 | 0.3846 | 0.4074 | 0.4286 | 0.4483 | 0.4561 | 0.4667 | 0.4839 |
| $j$ | 0.8750 | 0.8657 | 0.8571 | 0.8493 | 0.8475 | 0.8421 | 0.8354 | 0.8293 | 0.8235 | 0.8718 | 0.8642 | 0.8571 | 0.8506 | 0.8480 | 0.8444 | 0.8387 |
| $K$ | 131.3 | 157.0 | 183.7 | 211.2 | 218.2 | 239.3 | 268.1 | 297.3 | 327.0 | 167.7 | 193.6 | 220.4 | 247.9 | 261.1 | 275.9 | 304.4 |
| | $f_s = 18{,}000$ | | | | | | | | | $f_s = 18{,}000$ | | | | | | |
| $f_b$ | **800** | **900** | **1000** | **1100** | **1125** | **1200** | **1300** | **1400** | **1500** | **1000** | **1100** | **1200** | **1300** | **1350** | **1400** | **1500** |
| $p$ | 0.0077 | 0.0094 | 0.0111 | 0.0129 | 0.0134 | 0.0148 | 0.0168 | 0.0188 | 0.0208 | 0.0099 | 0.0116 | 0.0133 | 0.0151 | 0.0161 | 0.0170 | 0.0189 |
| $k$ | 0.3478 | 0.3750 | 0.4000 | 0.4230 | 0.4286 | 0.4444 | 0.4643 | 0.4828 | 0.5000 | 0.3571 | 0.3793 | 0.4000 | 0.4193 | 0.4286 | 0.4374 | 0.4545 |
| $j$ | 0.8841 | 0.8750 | 0.8667 | 0.8590 | 0.8571 | 0.8519 | 0.8452 | 0.8391 | 0.8333 | 0.8810 | 0.8736 | 0.8667 | 0.8602 | 0.8571 | 0.8542 | 0.8485 |
| $K$ | 123.0 | 147.7 | 173.3 | 199.9 | 206.6 | 227.2 | 255.1 | 283.6 | 312.5 | 157.3 | 182.3 | 208.0 | 234.4 | 248.0 | 261.5 | 289.2 |
| | $f_s = 20{,}000$ | | | | | | | | | $f_s = 20{,}000$ | | | | | | |
| $f_b$ | **800** | **900** | **1000** | **1100** | **1125** | **1200** | **1300** | **1400** | **1500** | **1000** | **1100** | **1200** | **1300** | **1350** | **1400** | **1500** |
| $p$ | 0.0065 | 0.0079 | **0.0094** | 0.0109 | **0.0113** | 0.0126 | 0.0142 | 0.0160 | 0.0178 | 0.0083 | 0.0098 | **0.0113** | 0.0128 | **0.0136** | 0.0144 | 0.0161 |
| $k$ | 0.3243 | 0.3506 | **0.3750** | 0.3976 | **0.4030** | 0.4186 | 0.4382 | 0.4565 | 0.4737 | 0.3333 | 0.3548 | **0.3750** | 0.3939 | **0.4030** | 0.4117 | 0.4286 |
| $j$ | 0.8919 | 0.8831 | **0.8750** | 0.8675 | **0.8657** | 0.8605 | 0.8539 | 0.8478 | 0.8421 | 0.8889 | 0.8817 | **0.8750** | 0.8687 | **0.8657** | 0.8628 | 0.8571 |
| $K$ | 115.7 | 139.3 | **164.1** | 189.7 | **196.2** | 216.1 | 243.2 | 270.9 | 299.2 | 148.1 | 172.1 | **196.9** | 222.4 | **235.5** | 248.7 | 275.5 |



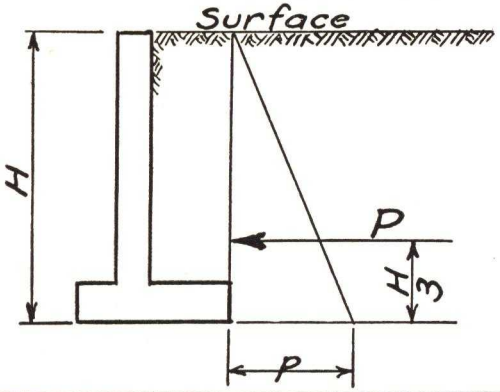

Curve showing distribution of load for rectangular slabs supported at the four edges.

*Figure 45*

## TABLE NO. 83

## EARTH PRESSURES AND MOMENTS FROM RANKINE'S FORMULA WITHOUT SURCHARGE

**Equivalent to a fluid pressure of 30 pounds per cubic foot**




$\varnothing$ = Angle of repose, assumed at 1½ to 1.

w = Unit weight of filling, assumed at 105 lbs. per cu ft.

H = Height of filling in feet.

p = Unit pressure at depth H, in lbs. per sq. ft.

$$p = wH\frac{1 - \text{Sin}\,\varnothing}{1 + \text{Sin}\,\varnothing} = 30H.$$

P = Total pressure per linear ft. = $15H^2$ lbs.

M = Moment per linear ft. = $5H^3$ ft. lbs.

| H | p | P | M | H | p | P | M |
|---|---|---|---|---|---|---|---|
| 4 | 120 | 240 | 320 | 36 | 1,080 | 19,440 | 233,280 |
| 5 | 150 | 375 | 625 | 37 | 1,110 | 20,535 | 253,265 |
| 6 | 180 | 540 | 1,080 | 38 | 1,140 | 21,660 | 274,360 |
| 7 | 210 | 735 | 1,715 | 39 | 1,170 | 22,815 | 296,595 |
| 8 | 240 | 960 | 2,560 | 40 | 1,200 | 24,000 | 320,000 |
| 9 | 270 | 1,215 | 3,645 | 41 | 1,230 | 25,215 | 344,605 |
| 10 | 300 | 1,500 | 5,000 | 42 | 1,260 | 26,460 | 370,440 |
| 11 | 330 | 1,815 | 6,655 | 43 | 1,290 | 27,735 | 397,535 |
| 12 | 360 | 2,160 | 8,640 | 44 | 1,320 | 29,040 | 425,920 |
| 13 | 390 | 2,535 | 10,985 | 45 | 1,350 | 30,375 | 455,625 |
| 14 | 420 | 2,940 | 13,720 | 46 | 1,380 | 31,740 | 486,680 |
| 15 | 450 | 3,375 | 16,875 | 47 | 1,410 | 33,135 | 519,115 |
| 16 | 480 | 3,840 | 20,480 | 48 | 1,440 | 34,560 | 552,960 |
| 17 | 510 | 4,335 | 24,565 | 49 | 1,470 | 36,015 | 588,245 |
| 18 | 540 | 4,860 | 29,160 | 50 | 1,500 | 37,500 | 625,000 |
| 19 | 570 | 5,415 | 34,295 | 51 | 1,530 | 39,015 | 663,255 |
| 20 | 600 | 6,000 | 40,000 | 52 | 1,560 | 40,560 | 703,040 |
| 21 | 630 | 6,615 | 46,305 | 53 | 1,590 | 42,135 | 744,385 |
| 22 | 660 | 7,260 | 53,240 | 54 | 1,620 | 43,740 | 787,320 |
| 23 | 690 | 7,935 | 60,835 | 55 | 1,650 | 45,375 | 831,875 |
| 24 | 720 | 8,640 | 69,120 | 56 | 1,680 | 47,040 | 878,080 |
| 25 | 750 | 9,375 | 78,125 | 57 | 1,710 | 48,735 | 925,965 |
| 26 | 780 | 10,140 | 87,880 | 58 | 1,740 | 50,460 | 975,560 |
| 27 | 810 | 10,935 | 98,415 | 59 | 1,770 | 52,215 | 1,026,895 |
| 28 | 840 | 11,760 | 109,760 | 60 | 1,800 | 54,000 | 1,080,000 |
| 29 | 870 | 12,615 | 121,945 | 61 | 1,830 | 55,815 | 1,134,905 |
| 30 | 900 | 13,500 | 135,000 | 62 | 1,860 | 57,660 | 1,191,640 |
| 31 | 930 | 14,415 | 148,955 | 63 | 1,890 | 59,535 | 1,250,235 |
| 32 | 960 | 15,360 | 163,840 | 64 | 1,920 | 61,440 | 1,310,720 |
| 33 | 990 | 16,335 | 179,685 | 65 | 1,950 | 63,375 | 1,373,125 |
| 34 | 1,020 | 17,340 | 196,520 | 66 | 1,980 | 65,340 | 1,437,480 |
| 35 | 1,050 | 18,375 | 214,375 | 67 | 2,010 | 67,335 | 1,503,815 |

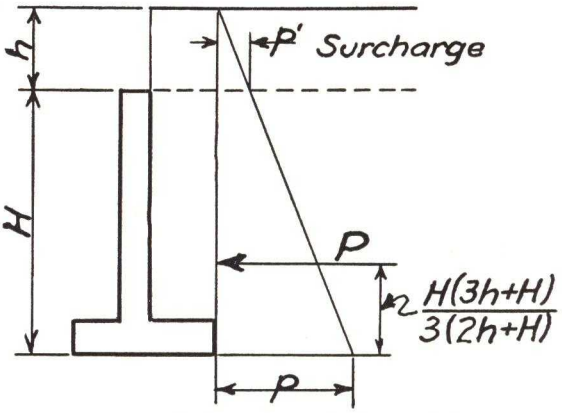
Reproduced from "Specifications for Design of Highway Structures", State of Ohio, Department of Highways, J. R. Burkey, Ch ef Engineer of Bridges. September, 1933.

## TABLE 84

## EARTH PRESSURES AND MOMENTS FROM RANKINE'S FORMULA WITH SURCHARGE

**Equivalent to a fluid pressure of 30 pounds per cubic foot**

$\varnothing$ = Angle of repose, assumed at 1½ to 1.
W = Unit weight of filling and surcharge. Assumed at 105 lbs. per cu. ft.
H = Height of filling in feet.
h = Height of surcharge in feet.
p = Unit pressure at depth (H+h) = 30 (H+h) lbs. per sq. ft.
p′ = Unit pressure at depth h = 30 h lbs. per sq. ft.
P. = Total pressure per linear ft. = 15H (2h+H) pounds.
M = Moment per linear foot = $5H^2$ (3h+H).




| H | h = 1 ft. | | h = 3 ft. | | h = 6 ft. | |
|---|---|---|---|---|---|---|
| | P | M | P | M | P | M |
| 4 | 360 | 560 | 600 | 1,040 | 960 | 1,760 |
| 5 | 525 | 1,000 | 825 | 1,750 | 1,275 | 2,875 |
| 6 | 720 | 1,620 | 1,080 | 2,700 | 1,620 | 4,320 |
| 7 | 945 | 2,450 | 1,365 | 3,920 | 1,995 | 6,125 |
| 8 | 1,200 | 3,520 | 1,680 | 5,440 | 2,400 | 8,320 |
| 9 | 1,485 | 4,860 | 2,025 | 7,290 | 2,835 | 10,935 |
| 10 | 1,800 | 6,500 | 2,400 | 9,500 | 3,300 | 14,000 |
| 11 | 2,145 | 8,470 | 2,805 | 12,100 | 3,795 | 17,545 |
| 12 | 2,520 | 10,800 | 3,240 | 15,120 | 4,320 | 21,600 |
| 13 | 2,925 | 13,520 | 3,705 | 18,590 | 4,875 | 26,195 |
| 14 | 3,360 | 16,660 | 4,200 | 22,540 | 5,460 | 31,360 |
| 15 | 3,825 | 20,250 | 4,725 | 27,000 | 6,075 | 37,125 |
| 16 | 4,320 | 24,320 | 5,280 | 32,000 | 6,720 | 43,520 |
| 17 | 4,845 | 28,900 | 5,865 | 37,570 | 7,395 | 50,575 |
| 18 | 5,400 | 34,020 | 6,480 | 43,740 | 8,100 | 58,320 |
| 19 | 5,985 | 39,710 | 7,125 | 50,540 | 8,835 | 66,785 |
| 20 | 6,600 | 46,000 | 7,800 | 58,000 | 9,600 | 76,000 |
| 21 | 7,245 | 52,920 | 8,505 | 66,150 | 10,395 | 85,995 |
| 22 | 7,920 | 60,500 | 9,240 | 75,020 | 11,220 | 96,800 |
| 23 | 8,625 | 68,770 | 10,005 | 84,640 | 12,075 | 108,445 |
| 24 | 9,360 | 77,760 | 10,800 | 95,040 | 12,960 | 120,960 |
| 25 | 10,125 | 87,500 | 11,625 | 106,250 | 13,875 | 134,375 |
| 26 | 10,920 | 98,020 | 12,480 | 118,300 | 14,820 | 148,720 |
| 27 | 11,745 | 109,350 | 13,365 | 131,220 | 15,795 | 164,025 |
| 28 | 12,600 | 121,520 | 14,280 | 145,040 | 16,800 | 180,320 |
| 29 | 13,485 | 134,560 | 15,225 | 159,790 | 17,835 | 197,635 |
| 30 | 14,400 | 148,500 | 16,200 | 175,500 | 18,900 | 216,000 |

Reproduced from "Specifications for Design of Highway Structures", State of Ohio, Department of Highways, J. R. Burkey, Chief Engineer of Bridges. September, 1933.

## TABLE NO. 85

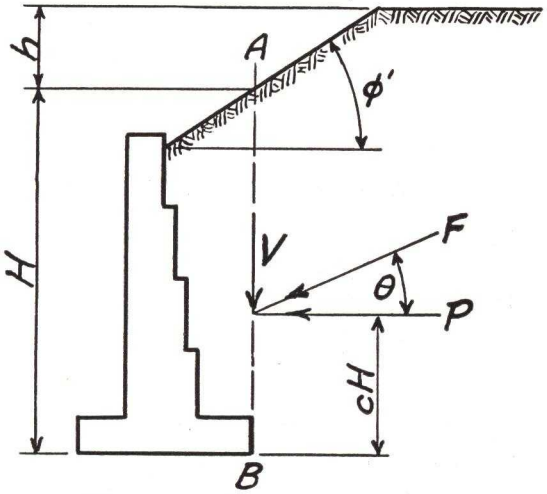
## FORMULAS FOR EARTH PRESSURES AND MOMENTS FOR SLOPING SURCHARGE TO A DEFINITE HEIGHT

**Equivalent to a fluid pressure of 30 pounds per cubic foot**

$\varnothing$ = Angle of repose. Assumed at 1½ to 1, or 33°-41′ from horizontal.
$\varnothing'$ = Angle of inclination of earth behind wall. Assumed at 1½ to 1.
w = Unit weight of filling. Assumed at 105 pounds per cu. ft.
H = Height of filling in feet, measured vertically at back of heel.
h = additional height above H, in feet, to which slope extends.
F = Total inclined pressure per linear foot of wall = $kH^2$ pounds.
$\theta$ = Angle of inclination of force F.
P = Horizontal component of F = F cos $\theta$ (pounds).
V = Vertical component of F = F sin $\theta$ (pounds).
Height at which force F is applied = cH (feet).
M = Overturning moment per lineal foot of wall about heel due to F, = $kH^2 \cos \theta \times cH = ckH^3 \cos \theta = KH^3$ (foot pounds).
K = ck cos $\theta$.

Note: For designing any part of wall above footing, vertical plane AB shall be moved over to back of wall, and heel shall not be considered.

Vertical component "V" shall be applied as indicated in sketch when computing overturning of wall but it may be considered as uniformly distributed over rear projection of footing for design of the heel.




| h/H | k | c | $\theta$ | K | Remarks |
|---|---|---|---|---|---|
| 0. | 15.00 | .333 | 0° | 5.00 | No Surcharge |
| 0.10 | 16.38 | .355 | 14°14′ | 5.64 | |
| 0.20 | 18.06 | .360 | 20°06′ | 6.11 | |
| 0.33 | 21.00 | .363 | 25°37′ | 6.87 | |
| 0.50 | 23.73 | .364 | 28°27′ | 7.59 | |
| 0.67 | 25.51 | .364 | 29°52′ | 8.05 | |
| 0.83 | 26.98 | .364 | 30°43′ | 8.44 | |
| 1.00 | 28.46 | .364 | 31°24′ | 8.84 | |
| 1.25 | 30.03 | .360 | 31°58′ | 9.17 | |
| 1.67 | 31.92 | .354 | 32°31′ | 9.53 | |
| 2.00 | 32.97 | .351 | 32°46′ | 9.73 | |
| 2.50 | 34.02 | .347 | 32°58′ | 9.90 | |
| 3.33 | 35.28 | .344 | 33°10′ | 10.16 | |
| 5.00 | 36.96 | .341 | 33°23′ | 10.52 | |
| 10.00 | 39.48 | .338 | 33°35′ | 11.12 | |
| ∞ | 43.69 | .333 | 33°41′ | 12.12 | Infinite Surcharge |

Reproduced from "Specifications for Design of Highway Structures", State of Ohio, Department of Highways, J. R. Burkey, Chief Engineer of Bridges. September, 1933.

## STANDARD SPECIFICATIONS

The following specifications for masonry materials have been summarized from the latest standard specification of the American Society for Testing Materials and Federal Specification Board. Complete copies of A.S.T.M. Specifications are usually available at the American Society for Testing Materials, 260 S. Broad St., Philadelphia, Pa. Federal specifications are for sale by the Superintendent of Documents, Washington, D. C.

Specifications for Glazed Brick and Tile may be obtained free of charge by addressing a request to the Glazed Brick and Tile Institute, affiliated with Structural Clay Products Institute, Washington, D. C.

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## SUMMARY OF FEDERAL SPECIFICATION

FOR

# BRICK; BUILDING (COMMON), CLAY

## SS—B—656

JUNE 28, 1932

### A. APPLICABLE FEDERAL SPECIFICATIONS

**A-1.** There are no other Federal specifications applicable to this specification.

### B. GRADES

**B-1.** This specification is applicable to common, solid, or hollow (not face) clay brick of any of the following three grades:

H. Hard
M. Medium
S. Soft

### C. MATERIALS AND WORKMANSHIP

**C-1.** Brick under this specification shall be of clay or shale, be sound, of compact structure, reasonably uniform in shape, free from stones and pebbles that would affect their serviceability or strength, and without excessive laminations or warpings.

### D. GENERAL REQUIREMENTS

**D-1.** The size of brick shall be as specified in the invitation for bids with permissible maximum variations from the specified size, over or under, of one-eighth inch in breadth or depth, and one-quarter inch in length.

**D-2.** Bricks shall be delivered in good condition, with not more than 5 per cent broken bricks.

**D-3.** At the completion of the absorption test the bricks shall show no evidence of disintegration.

### E. DETAIL REQUIREMENTS

**E-1.** The bricks shall meet the following absorption and strength requirements for their respective grade. The standing of any set of bricks shall be determined by the requirement which gives it the lowest grade. Unless otherwise specified in the invitation for bids, medium (M) or hard (H) brick shall be accepted in lieu of soft (S) brick, and hard (H) brick in lieu of medium (M) brick.

**E-2. Physical requirements—**

| Grade | Maximum absorption | | Modulus of rupture | |
|---|---|---|---|---|
| | Average of 5 | Individual, maximum | Average of 5 | Individual, minimum |
| | | | Lbs./in.$^2$ | Lbs./in.$^2$ |
| H | 10 per cent or less..... | 12 per cent..... | 600 or more | 400 |
| M | 16 per cent..... | 20 per cent..... | 450 to 600 | 300 |
| S | No limit..... | No limit..... | 300 to 450 | 200 |

**E-3.** The absorption requirement may be waived by the purchasing officer.

**E-4.** If the bricks are for facing and a particular surface appearance is desired, the color and texture will be specified.

## F. METHODS OF SAMPLING, INSPECTION, AND TESTS

**F-1. Sampling.** — Ten bricks selected by the inspector, so as to be fairly representative of a quantity not exceeding 50,000 bricks shall constitute a sample. If the bricks are delivered by car or boat, one sample shall be taken from each carload or boatload. If the bricks are delivered by truck or wagon, one or more samples shall be taken at the point of origin, covering all of the material from which shipments are to be made. Additional representative samples may be taken at any time or place at the discretion of the inspector.

**F-2. Tests.** — The sample of 10 bricks shall be dried to constant weight at a temperature of 212° to 220° F.

**F-2a. Absorption.** — When cool five bricks shall be weighed separately on scales sensitive to within two-tenths of 1 per cent of the weight. They shall then be completely immersed in soft, distilled, or rain water at room temperature (60° to 80° F.) for five hours. They shall then be removed from the water and again weighed separately after wiping the surfaces with a damp cloth. The difference between the weight after immersion and the weight when dry, multiplied by 100 and divided by the dry weight shall be taken as the absorption in per cent.

**F-2b. Transverse strength.** — The other five bricks of the sample, previously dried, shall be tested flatwise by supporting on a span of 7 inches and applying the load at mid-span. The loads shall be applied by means of a standard testing machine or a calibrated portable or semiportable testing equipment. A steel bearing plate about one-fourth inch thick by 1½ inches wide shall be placed between the upper knife-edge and the brick. The knife-edges in contact with the brick shall be mounted so they will adjust themselves to the irregularities in the shape of the brick, and one or both of the lower bearings shall be free to follow any movement of the brick during the test.

The modulus of rupture of each brick shall be calculated by means of the following formula:

$$R = \frac{3WL}{2bd^2}$$

where

W = the total load in pounds at which the brick failed,
L = the distance between the supports in inches,
b = the width of the brick in inches,
d = the depth of the brick in inches, and
R = modulus of rupture in pounds per square inch.

## G. PACKAGING, PACKING, AND MARKING

**G-1.** Shall be in accordance with commercial practice unless otherwise specified.

# SUMMARY OF
# A.S.T.M. TENTATIVE SPECIFICATIONS
## FOR
# BUILDING BRICK (MADE FROM CLAY OR SHALE)
## A.S.T.M. Designation: C62 - 37T

### SCOPE

1. (a) These specifications cover brick made from clay or shale and burned, and which are intended for use in brick masonry. Three grades of brick are covered, as follows:

Grade SW — Bricks intended for use where exposed to temperature below freezing in the presence of moisture.

> NOTE: As a typical example, bricks used for foundation courses and parapets in the northeastern quarter of the United States should conform to this grade.

Grade MW — Bricks intended for use where exposed to temperature below freezing but unlikely to be saturated with water.

> NOTE: As a typical example, bricks exposed in the face of the wall other than parapet or foundations, or bricks intended for structures located in regions of the United States characterized by less severe frost action or by drier climate than is found in the northeastern quarter of the United States, should conform to this grade.

Grade NW — Bricks intended for use as back-up or interior masonry, or if exposed, for use where no frost action occurs, or if frost-action occurs where the average annual precipitation is less than 15 in.

(b) Where bricks having strengths greater than called for by these specifications are required, the purchaser should specify the desired minimum compressive strength.

### PROPERTIES

2. (a) The bricks shall conform to the physical requirements for the grade specified, as follows:

| | Compressive Strength (bricks flatwise), lb. per sq. in., average gross area | | Water Absorption by 5-hr. Boiling per cent | | | Saturation Coefficient (C/B Ratio) * |
|---|---|---|---|---|---|---|
| | Average of 5 Bricks | Individual Minimum | Average of 5 Bricks | Individual Maximum | Average of 5 Bricks | Individual Maximum |
| Grade SW.... | 2500 | 2500 | 20 | 20 | 0.80 | 0.80 |
| Grade MW... | 2500 | 2000 | 20 | 25 | 0.85 | 0.90 |
| Grade NW.... | 1500 | 1250 | no limit | no limit | no limit | no limit |

*The saturation coefficient (C/B ratio) is the ratio of absorption by 24-hr. submersion in cold water to that after 5-hr. submersion in boiling water.

(b) Unless otherwise specified by the purchaser, bricks of Grades SW and MW shall be accepted in lieu of Grade NW, and Grade SW in lieu of Grade MW.

(c) If the average compressive strength is greater than 8000 lb. per sq. in. or the average water absorption is less than 8 per cent by weight after 5 hr. submersion in boiling water or less than 6.4 per cent after 24-hr. submersion in cold water, the requirement for C/B ratio shall be waived.

## TEST SPECIMENS

3. For purpose of tests, bricks representative of the commercial product shall be selected by a competent person appointed by the purchaser, the place or places of selection to be designated when the purchase order is placed. The manufacturer or seller shall furnish specimens for tests without charge. All bricks shall be carefully examined and their condition noted before testing. For the purpose of tests, not less than five brick shall be required for each investigation. In general, two samples of five brick each shall be tested for each 100,000 bricks contained in the lot under consideration; but where the total quantity exceeds 500,000, one sample of five brick from each 100,000 shall be tested. Additional representative samples may be taken at any time or place at the discretion of the purchaser.

## STANDARD SIZES

4. The standard sizes of brick made from clay or shale shall conform to the following dimensions, within permissible variations of $\pm\frac{1}{16}$ in. in depth, $\pm\frac{1}{8}$ in. in width, and $\pm\frac{1}{4}$ in. in length:

| Type | Depth in. | Width in. | Length in. |
|---|---|---|---|
| Common brick | 2¼ | 3¾ | 8 |
| Rough-face brick | 2¼ | 3¾ | 8 |
| Smooth-face brick | 2¼ | 3⅞ | 8 |

## WORKMANSHIP AND FINISH

5. When any particular surface appearance or uniformity is desired, it shall be as mutually agreed upon by the manufacturer and the purchaser.

## VISUAL INSPECTION

6. All brick shall be free from cracks, warpage, stones, pebbles, or particles of lime that would affect their serviceability or strength.

## METHODS OF TESTING

7. The bricks shall be tested in accordance with the Standard Methods of Testing Brick (Compressive Strength and Absorption) (A.S.T.M. **Designation:** C67) of the American Society for Testing Materials.

# SUMMARY OF
# A.S.T.M. TENTATIVE SPECIFICATIONS
## FOR
# SEWER BRICK (MADE FROM CLAY OR SHALE)
## A.S.T.M. DESIGNATION: C32-37T

**SCOPE:**

1. These specifications cover brick made from clay or shale and burned, to be used in drainage structures for the conveyance of sewage, industrial wastes, and storm water. Three grades of brick are covered as follows:

Grade SA — Brick intended for use in structures requiring imperviousness and resistance to the action of sewage carrying large quantities of abrasive material at velocities exceeding 8 ft. per sec.

Grade MA — Brick intended for use in structures requiring imperviousness and resistance to the action: (a) of sewage free from abrasive materials, and (b) of sewage carrying abrasive materials at velocities of 8 ft. per sec. or less.

Grade NA — Brick intended for use in structures not requiring high degrees of imperviousness nor of abrasive resistance.

> **Note** — Bricks of Grade NA are suitable for use in catch basins, arches, the upper portions of manholes, and for backing.

**PROPERTIES:**

2. (a) The brick shall conform to the physical requirements for the grades specified, as follows:

| | Compressive strength, lb. per sq. in., average gross area | | Water absorption by 5-hr. boiling, per cent | |
|---|---|---|---|---|
| | Average of 5 bricks | Individual Minimum | Average of 5 bricks | Individual Maximum |
| Grade SA | 8,000 | 5,000 | 6 | 9 |
| Grade MA | 5,000 | 3,000 | 12 | 16 |
| Grade NA | 2,500 | 2,000 | 20 | 24 |

(b) Unless otherwise specified by the purchaser, bricks of Grade SA shall be accepted in lieu of Grade MA; also Grades SA and MA shall be accepted in lieu of Grade NA.

**TEST SPECIMENS:**

3. (a) For purpose of tests, bricks representative of the commercial product shall be selected by a competent person appointed by the purchaser, the place or places of selection to be designated when the purchase order is placed. The manufacturer or seller shall furnish specimens for tests without charge. All bricks shall be carefully examined and their condition noted before testing.

(b) For the purpose of tests, not less than five brick shall be required for each investigation. In general, two samples of five brick each shall be tested for each 100,000 bricks contained in the lot under consideration; but where the total quantity exceeds 500,000, one sample of five brick from each 100,000 shall be tested. Additional representative samples may be taken at any time or place at the discretion of the purchaser.

**SIZES:**

4. Brick of one or more of the following designated sizes as specified shall be furnished:

| Size | Depth in. | Width in. | Length in. |
|---|---|---|---|
| No. 1 | 2¼ | 3¾ | 8 |
| No. 2 | 2½ | 4 | 8½ |
| No. 3 | 3 | 4 | 8½ |
| No. 4 | 3½ | 4 | 8½ |

**PERMISSIBLE VARIATIONS IN SIZE:**

5. For any lot of sewer brick furnished under these specifications, not more than 2 per cent of the bricks shall vary from the nominal size requirements specified in Section 4 by more than ± ⅛ in. in either transverse dimension, or by more than ± ¼ in. in length.

**MATERIALS AND WORKMANSHIP:**

6. (a) The brick shall be manufactured from surface clay, fire clay, or shale, or a combination of these materials. The composition shall be such that the finished brick are free from deleterious materials including coarse grains, stones, pebbles, or visible grains of lime.

(b) Brick shall be of rectangular cross-section with substantially straight, square corners. The ends and at least one edge shall have plain surfaces.

(c) When paving brick are used, lugs and grooves will be permitted on one edge and the ends. Kiln marks not exceeding 3/16 of an inch in depth shall be permitted on the opposite edge.

(d) Brick shall be free from checks or cracks extending into the body of the brick in a manner that would affect their serviceability or strength.

> NOTE: Where cored brick are permitted by the purchaser, they shall conform to the requirements of these specifications.

**METHODS OF TESTING:**

7. Brick shall be tested in accordance with the Standard Methods of Testing Brick (Modulus of Rupture, Compressive Strength, Absorption) (A.S.T.M. Designation: C67 of the American Society for Testing Materials).

# SPECIFICATIONS
## FOR
# BRICK MASONRY BUILDINGS AND APPURTENANT STRUCTURES

### 1. BRICK

All brick shall be sound, whole, new clay or shale brick. Brick used for parapet walls, garden walls, or in masonry below grade or in contact with earth, such as foundations, retaining walls, walks, terraces, etc., shall conform to the Standard Specifications for Building Brick, Grade SW, A.S.T.M. C62-37T.

Brick used as facing for exterior walls exposed to the weather on one side only and not in contact with earth, shall conform to Standard Specifications for Building Brick, Grade MW, A.S.T.M. C62-37T. Brick used as back-up in walls not in contact with earth or in interior walls or partitions, shall conform to Standard Specifications, Grade NW, A.S.T.M. C62-37T.

NOTE: The above recommendations are, in general, applicable to exposures encountered in the Northeastern quarter of the United States. In regions characterized by less severe frost action or by dryer climate than is found in the Northeastern quarter of the United States, Grade MW brick may be specified in lieu of Grade SW.

### 2. MORTAR

See Chapter 2 for specifications for mortar and mortar materials.

### 3. WORKMANSHIP

All brick shall be laid plumb to lines. All mortar joints shall be full. Mortar beds shall be spread smooth or only slightly furrowed. The ends of the brick shall be buttered with sufficient mortar to completely fill the end joint when the brick is placed, and the vertical longitudinal joints shall be completely filled. Vertical longitudinal (collar) joints shall be filled by parging the facing or back-up, by pouring the vertical joint full of grout, by the pick and dip method or by shoving. Slushing of joints after laying should not be necessary. Closures may be rocked into place with head joints thrown against the two adjacent brick in place.

### 4. WETTING BRICK

All brick having rates of absorption in excess of 20 grams per minute when placed flat side down in $\frac{1}{8}''$ of water, shall be wet sufficiently so that the rate of absorption when laid does not exceed 20 grams per minute, except brick to be used with grouted masonry for which the rate of absorption may be increased to 30 grams per minute. Brick should be wetted from 3 to 4 hours before they are to be used, and shall be free from water adhering to the surface of the brick when they are placed into the wall.

### 5. BONDING

At least every sixth course of brickwork shall be a full header course or there shall be at least one header per 72 sq. in. of wall surface, except that masonry headers will not be required where grouted masonry is used and all interior joints are filled with grout.

### 6. CLEANING

Exterior brickwork shall be wiped with a damp cloth as the work progresses and thoroughly cleaned on completion. If stiff brushes and water do not suffice, the surface shall be thoroughly wetted first with plain water and then a solution of not more than 1 part hydrochloric (muriatic) acid to ten parts water (preferably 1 part to 20 of water), shall be scrubbed on (taking care to protect all sash, metal lintels, etc.) and immediately rinsed off thoroughly with clean water.

## 7. JOINTS

Exterior joints below grade shall be trowel pointed and elsewhere shall be tooled with a round jointer just before the initial set occurs, compacting the mortar into the joint and against the bricks with firm pressure.

NOTE: Stripped, raked or struck joints are not recommended.

## 8. CAULKING

Outside joints at the perimeter of exterior door and window frames should not be less than ¼″ nor more than ⅜″ wide and shall be cleaned out to a uniform depth of at least ¾″ and filled solid with an approved elastic caulking compound forced into place with a pressure gun. Caulking compound shall be elastic and waterproof and shall gradually form a thin, tough skin on exposed surfaces while remaining plastic indefinitely beneath the surface. It shall not be affected by long exposure to extreme of outside temperature.

## 9. DAMPPROOFING

Dampproofing shall consist of one coat of creosote oil and two coats of coal tar pitch. The pitch shall be heated to flow freely, but not above 350 degrees F. Dull or porous spots appearing after application shall be remopped with pitch. Dampproofing shall be applied only to dry and reasonably smooth surfaces, and all cracks, holes, etc., shall be pointed before application.

Dampproofing shall be applied to the outside of all basement walls in contact with earth, and shall extend from the outer edge of footings to 4″ above finished grade, and to the level of underside of area walls. The dampproofing shall extend at least 12″ onto area, porch or garage walls. The dampproofing shall be covered with a ½″ coat of 1:3 portland cement mortar, trowelled smooth, beveled at top and coved out to edge of footing.

NOTE: The above specifications for dampproofing will be adequate for the average soil condition. However, where masonry is constructed in water-bearing soils, membrane waterproofing may be necessary to prevent the penetration of moisture. Where membrane waterproofing is specified, the manufacturer's recommended method of application should be followed.

## 10. CAVITY WALLS

Cavity walls shall be constructed as indicated on the plans. Upon the completion of the work, all mortar droppings shall be removed from the cavity and weep holes at the bottom of the cavity shall be open to permit the escape of moisture. Metal ties, dampproofing, flashing, and closure courses shall be installed as indicated on the detail sheets.

## 11. BRICK VENEER

Brick veneer shall be installed as indicated on the plans. Where veneer is to be applied to a frame or stucco building, an adequate foundation for the veneer shall be constructed as indicated on the plans. Workmanship shall conform to the requirements of this specification and metal ties, brick sills, flashing and dampproof courses shall be installed as indicated on the detail sheets.

## 12. PARAPET WALLS

Wherever the plans indicate a parapet wall, such walls shall be built of brick known to resist weathering. The corners of parapets shall have not less than one ¼″ round rod in every third joint extending at least three feet on each side of the corner. If the parapet is more than 50 feet in length, these ¼″ rods shall be continuous for the entire length of the parapet. Coping shall consist of salt glazed tile with lapped and sealed joints.

## 13. JOINING OF WORK

Where fresh masonry joins masonry that is partially set or totally set, the exposed surface of the set masonry shall be cleaned, roughened, and lightly wetted so as to obtain the best possible bond with the new work. All loose bricks and mortar shall be removed.

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# APPENDIX

## GLOSSARY OF TERMS

**ABSORPTION (of brick)** — Is obtained by immersion in either cold or boiling water for stated periods of time. It is usually expressed as a percent of the weight of the dry brick.

**ABUTMENT** — That part of a structure which takes the thrust of a beam, arch, vault, truss or girder.

**ADOBE BRICK** — A large clay brick, of varying size, roughly molded and sun dried. In certain sections of this country, a brick approximating paving brick size is known by this term.

**ANCHOR** — A piece or connected pieces of metal used for tying together two or more pieces of masonry materials. See cramp, wall plate, and swedge anchor.

**APRON WALL** — That part of a panel wall between the window sill and the support of the panel wall.

**ARCH** — A curved structural member used to span an opening or recess; also built flat. (See supplementary definitions under kind.) Structurally, an arch is a piece or assemblage of pieces so arranged over an opening that the supported load is resolved into pressures on the side supports, and practically normal to their faces.

**ARCH (Back)** — A concealed arch carrying the backing or inner part of a wall where the exterior facing material is carried by a lintel or other form of support.

**ARCH (Basket Handle)** — A three centered arch.

**ARCH (Cambered)** — A flat arch with a slightly concave intrados.

**ARCH (Catenary)** — An arch whose intrados or central line is a catenary curve.

**ARCH (Cusped)** — One which has cusps or foliations worked on the intrados.

**ARCH (Cycloidal)** — One whose intrados or center line is laid out to a cycloidal curve.

**ARCH (Flat)** — One having a horizontal or nearly horizontal intrados and, in most cases, a horizontal extrados.

**ARCH (Haunched)** — One in which the curve at the crown differs from that at the haunches.

**ARCH (Horseshoe)** — One in which the curves are carried below the springing (also spring) line so that the opening at the bottom of the arch is less than its greatest span. A Moorish arch.

**ARCH (Inverted)** — One whose springing line is above the intrados and the intrados above the extrados.

**ARCH (Jack)** — Same as a flat arch. Also any arch doing rough work or one roughly built.

**ARCH (Ogee)** — One having a reversed curve at the point.

**ARCH (Pointed)** — One in which two curves meet at the crown at an angle, more or less acute. Ordinary two centered pointed arches are called lancet, acute, equilateral or blunt arches.

**ARCH (Rampant)** — One in which the impost on one side is higher than on the other side.

**ARCH (Relieving)** — One built over a lintel, or other similar closure or opening in a wall, and intended to divert the super-imposed load above the opening to the piers or abutments on both sides, thus relieving the lintel or flat arch from excessive loading. Also known as a discharging arch.

**ARCH (Rowlock)** — One in which the brick or other pieces of solid masonry material are arranged in separate concentric rings.

**ARCH (Round)** — One of semi-circular curvature and usually limited to one only slightly stilted, if at all, above the imposts, so that its general appearance is that of a semicircle.

**ARCH (Safety)** — Same as a relieving or discharging arch.

**ARCH (Segmental)** — One in which the centers are below the angle made by the impost with the inner face of the abutment.

**ARCH (Skew)** — One in which the axis and barrel form an oblique angle with the face of the wall.

**ARCH (Splayed)** — An arch opening with a larger radius at one side of a wall than on the other side.

**ARCH (Stilted)** — One in which the impost and architectural mouldings, etc., are notably lower on one side than on the other.

**ARCH (Straight)** — Same as a flat arch.

**ARCH (Straining)** — One used as a strut; as in a flying buttress.

**ARCH (Trimmer)** — An arch, usually of brickwork and of low rise, built between trimmers where a floor is framed around a fireplace or chimney breast and used for supporting the hearth.

**AREA WALL** — The masonry surrounding or partly surrounding an area. It also serves as a retaining wall.

**BACK FILLING** — Rough masonry built in behind the facing, or between two faces; similar material or earth used in filling over the extrados of an arched construction. Also brickwork used to fill in the space between studs in a frame building.

**BACKUP** — That part of a masonry wall behind the exterior facing and consisting of one or more withes or thicknesses of brick or other masonry material.

**BACKING UP** — The operation of building up that part of a piece of masonry, other than its facing.

**BARGE COURSE** — A course of brick, forming the coping of a wall, which is set on edge and transversely to the wall.

**BASE** — The lowest part, or the lowest main division, of a building, column, pier or wall.

**BASE COURSE** — The lowest course of masonry of a wall or pier. A footing course.

**BAT** — A piece of broken brick.

**BEARING** — That part of a lintel, beam, girder or truss, which rests upon a column, pier or wall.

**BEARING PLATE** — A piece of steel, iron, or other material which receives the load concentration and transmits it to the masonry.

**BED** — The prepared soil, or layer of mortar, on or in which a piece of masonry material is laid.

**BEARING WALL** — A wall which supports any load in addition to its own weight.

**BELT COURSE** — Same as a string course.

**BLANK WALL** — One having no door, window or other opening.

**BLOCKING** — A method of bonding two adjoining or intersecting walls, not built at the same time, by means of offsets and overhanging blocks consisting of several courses of brick each.

**BLOCK (Hollow)** — A shape made of clay, terra cotta or other material fashioned with one or more openings in its body for lightness, etc., whose net sectional area does not exceed 75% of its gross sectional area.

**BOND** — The tying or bonding of the various pieces and parts of a masonry wall, by laying one piece across two or more pieces; the entire system of bonding or breaking joints as used in masonry construction. For description of various kinds of bonds, see text herewith. The mortar between brick is sometimes termed a bond.

**BOND (Course)** — The header course.

**BRICK** — A structural unit of burnt clay or shale, formed while plastic into a rectangular prism, usually solid, the net sectional area of which is not less than 75% of the gross sectional area.

**BRICK (Angle)** — Any brick shaped to an oblique angle to fit a salient corner.

**BRICK (Arch)** — A wedge-shaped brick for special use in the voussoir of an arch; also a brick from an arch of a kiln which has been hard-burned and is therefore regarded as more suitable for certain kinds of brickwork.

**BRICK (Clinker)** — A very hard-burned brick, so-called from its appearance and from the metallic sound when struck.

**BRICK (Common)** — Any brick made primarily for building purposes and not especially treated for texture or color, but including clinker and over-burnt brick.

**BRICK (Compass)** — Same as Arch Brick.

**BRICK (Fire)** — One made of refractory clay which will resist high temperatures.

**BRICK (Furring)** — A hollow brick used for furring or lining the inside face of a wall, usually the size of an ordinary brick and grooved or scored on the faces to afford a key for plastering.

**BRICK (Gauged)** — A brick which has been ground or otherwise prepared to accurately fit a given curve. An Arch Brick.

**BRICK (Hollow)** — A brick having one or more longitudinal perforations forming continuous ducts or channels, and having a net sectional area of 75% or less of the gross sectional area.

**BRICK (Salmon)** — A relatively soft, under-burned brick.

**BRICK (Facing)** — A brick made especially for facing purposes, usually treated to produce surface texture or made of selected clays or otherwise treated to produce the desired color.

**BRICK NOGGING** — The filling of brickwork between members of a framed wall or partition.

**BRICK AND BRICK** — A method of laying brick whereby the brick are laid touching each other with only mortar enough to fill the irregularities of the surface.

**BRICKWORK** — Any structure or structural part, made of brick and mortar.

**BUTTERING** — Placing mortar on a brick with a trowel before brick is laid.

**BUTTRESS** — A piece of masonry, like a pier, built against and bonded into a wall to strengthen the wall against side thrust.

**BUTTRESS (Flying)** — A detached buttress or pier of masonry, at some distance from the wall, and connected thereto by an arch or portion of an arch, so as to assist in resisting side thrust.

**CAISSON** — A foundation pier, either circular or rectilinear in plan, usually sunk to rock either by means of gravity, compressed air or by the open-well method.

**CAMBER** — A slight or upward curve of a structural member so that it becomes horizontal, or nearly so, when loaded.

**CAULKING** — The operation or method of rendering a joint tight against water by means of some plastic substances such as oakum and pitch, elastic cement, etc.

**CENTER** — A timber framework or mold upon which the masonry of an arch, vault or beam is supported until self-supporting.

**CHIMNEY ARCH** — The arch over the opening of a fireplace.

**CHIMNEY BACK** — The back of a fireplace against which the fire is built.

**CHIMNEY BAR** — The lintel above the fireplace opening used for supporting the brickwork above.

**CHIMNEY BREAST** — The front portion of a chimney which projects into the building from the face of the wall.

**CLOSER** — The last brick laid in a course; the end brick of a part of a course, fitted at the openings. A closer may be a whole brick or less in size.

**C/B RATIO** — The ratio of the weight of water absorbed by cold immersion (usually 24 hours) to the weight absorbed by immersion in boiling water (usually 5 hours). This ratio is also known as the saturation coefficient.

**COLUMN** — A pillar or pier of rather slender proportions which carries a load and acts as an upright support.

**COMMON BRICK** — See Brick (Common).

**COPING** — The material or member used to form a capping or finish on top of a wall, pier, etc., to protect the masonry below by throwing off the water to one or more sides.

**CORBEL** — That part of the masonry built outward from the face of masonry by projecting successive courses of the masonry.

**COUNTERFORT** — A buttress or portion projecting from a wall and upward from the foundation to provide additional resistance to thrusts.

**COURSE** — One of the continuous horizontal layers of masonry units which are bonded together and thus form a masonry structure.

**CRAMP** — An anchor for masonry, made of a short, flat bar of metal, with both ends turned down at right angles, and used for tying the masonry together by bedding the bent ends in holes provided in the masonry units.

**CROWN** — The top or head of an arch or vault.

**CURTAIN WALL** — A non-bearing wall built between columns or piers for the enclosure of a building, but not supported at each story.

**DAMP COURSE** — A course or layer of impervious material in a wall or floor to prevent the entrance of moisture from the ground or from a lower course.

**DEFORMED BARS** — Reinforcing bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling so as to firmly engage the surrounding mortar. Wire mesh with welded intersections not farther apart than twelve inches in the direction of the principal reinforcement and with cross wires not smaller than No. 10 may be rated as a deformed bar.

**DRIP** — Any projecting piece of material, member or part of a member so shaped and placed as to throw off water and prevent its running down the face of a wall or other surface of which it is a part.

**EFFECTIVE AREA OF BRICK MASONRY** — The area of a section which lies between the centroid of the tensile reinforcement and the compression face of the structural member.

**EFFECTIVE AREA OF REINFORCEMENT** — The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and that for which the effectiveness of the reinforcement is to be determined.

**EFFECTIVE DEPTH** — The distance from the center of gravity of tensile reinforcement to the compression surface of a structural member.

**EFFLORESCENCE** — A whitish powder, sometimes formed on the surface of masonry by deposition of soluble salts.

**ENCLOSURE WALL** — An exterior non-bearing wall in skeleton construction, anchored to columns, piers or floors, but not necessarily built between columns or piers nor wholly supported at each story.

**EXTERIOR WALL** — Any outside wall or vertical enclosure of a building other than a party wall.

**EXTRADOS** — The upper or convex surface of the voussoirs composing an arch or vault.

**FACE** — The front or exposed surface of a wall.

**FACING BRICK** — The same as Brick (Facing).

**FACING** — Any material, forming a part of the wall, used on the exterior as a finishing surface.

**FACED WALL** — A wall in which the facing and backing are so bonded with masonry as to exert common action under load.

**FIRE DIVISION WALL** — Any wall which subdivides a building so as to resist the spread of fire, but is not necessarily continuous through all stories to and above the roof.

**FIRE WALL** — Any wall which subdivides a building so as to resist the spread of fire, by starting at the foundation and extending continuously through all stories to and above the roof.

**FIRE-RESISTIVE MATERIAL** — See "Incombustible Material", improperly called "Fireproof Material."

**FIREPROOFING** — Any material or combination of materials used to enclose structural members so as to make them fire resistive.

**FIRE STOP** — Any piece or mass of fire resistive material used for filling in the open spaces between wood framework or to close other open parts of a structure in order to prevent the passage of fire.

**FOOTING** — That part of a foundation wall resting on the bearing soil, rock, piling, etc., which transmits the superimposed load to the bearing material.

**FOUNDATION** — That part of a load-bearing wall, below the level of the adjacent grade, or below the level of the first tier of floor beams or joists; also that part of a column which transmits the super-imposed load to the supporting soil, rock, piling, etc.

**FOUNDATION WALL** — That portion of a load-bearing wall below the level of the adjacent grade, or below the first tier of floor beams or joists, which transmits the superimposed load to the footing.

**GROUT** — A mixture of cementitious material (cement, lime), sand and sufficient water to make a consistency that will flow without separation of ingredients.

**HAUNCH** — The side of an arch about half-way between the crown and the abutment.

**HEADER** — A brick laid lengthwise across a wall and serving as a bond.

**HEADING COURSE** — A course of headers.

**HOLLOW WALL** — A wall built of solid masonry units laid in and so constructed as to provide an air space within the wall.

**IDEAL WALL** — A hollow wall of solid brick, with one or both tiers of brick on edge and using solid brick bonding headers.

**INCLOSURE WALL** — Same as "Enclosure Wall."

**INCOMBUSTIBLE (Building Material)** — Any building material which contains no matter subject to rapid oxidation within the temperature limits of a standard fire test of not less than 2½ hours duration. Note — Materials which continue burning after this time period are combustible.

**INTERIOR WALL** — Any wall entirely surrounded by the exterior walls of a building.

**INTRADOS** — The under or concave surface of an arch. The soffit.

**INVERT** — The upper or concave surface of an inverted arch construction. The bottom half of a sewer.

**KEY** — A wedge section of masonry placed at the crown of an arch, acting as a key.

**LACING COURSE** — A course of brick, or several adjacent courses considered collectively, inserted at frequent intervals, as in a stone wall as a bond course.

**LEAD** — The building up and racking back, on successive courses, at a corner of a building at the time a wall is started.

**LINTEL** — A beam of suitable material placed over a wall opening to carry the superimposed weight above.

**LINTEL (Safety)** — A lintel of wood or other suitable material placed behind the main lintel or behind an arch; generally used in conjunction with a relieving arch.

**MASONRY** — Brick, stone, tile, or other similar building units or materials, or combinations of them, bonded together with mortar.

**NON-BEARING WALL** — Any wall which carries no load other than its own weight.

**PANEL WALL** — A non-bearing wall in skeleton construction, built between columns or piers, and wholly supported at each story.

**PARAPET WALL** — A dwarf wall or barrier extending above the roof.

**PARTITION** — An interior wall, usually non-bearing, not exceeding one story in height.

**PARTY WALL** — A wall used, or adapted for use for joint service by adjoining buildings.

**PIER** — Any isolated mass or column of masonry.

**PILASTER** — An engaged pier, built as an integral part of a wall, and projecting slightly from either vertical surface thereof.

**POINTING** — Pushing mortar into a joint after a brick is laid.

**RACKING** — The methods of building the end of a wall so that it can be built onto and against without toothers.

**RAGGLE** — A groove or channel made in a mortar joint, or in the solid masonry material, to receive roofing, metal flashing or other material which is to be sealed in the masonry.

**RETAINING WALL** — Any wall designed to resist lateral pressure.

**RETURN** — Any surface turned back from the face of a principal surface.

**REVEAL** — That portion of a jamb or recess which is visible from the face of a wall back to the frame placed between the jambs.

**R-B-M** — An abbreviation for reinforced brick masonry.

**REINFORCED BRICK MASONRY (R-B-M)** — Brick masonry in which metal is imbedded in such a manner that the two materials act together in resisting forces.

**REINFORCEMENT** — Structural Steel Shapes, steel bars, rods, wire mesh or expanded metal imbedded or encased in brick masonry to increase the strength of the masonry.

**ROWLOCK** — A brick laid on its edge. Frequently spelled Rolok.

**SALMON** — See Brick (Salmon).

**SATURATION COEFFICIENT** — See C/B Ratio.

**SET IN** — The amount that the lower edge of a brick on the outside face of a wall is held back from the line of the top edge of the brick directly below it. Also called "set-off".

**SILL COURSE** — See String Course.

**SKELETON CONSTRUCTION** — A type of building construction in which all loads are transmitted to the foundation by a rigidly connected framework of suitable material, with the enclosing walls supported by girders or by the floor at each floor level.

**SKEW** — Inclination in any direction.

**SKEW BACK** — That portion of an abutment having an inclined face, which is arranged to receive the thrust of an arch.

**SOFFIT** — The under side of an arch, floor, lintel, stair or other similar construction.

**SOLDIER** — A brick laid on its end, so that its greatest dimension is vertical.

**SOLID WALL** — A wall built of solid masonry units, laid contiguously, with the spaces between the units completely filled with mortar. Also walls built of solid concrete.

**SPALL** — A small fragment removed from the face of stone, brick or other masonry material by a blow or by the action of the elements.

**SPAN** — The distance between two consecutive supports.

**SPANDREL WALL** — That part of a panel wall above the top of a window in one story and below the sill of the window in the story above. Also the space included between the extrados of two adjoining arches and a line approximately connecting their crowns.

**SPRINGING LINE** — A line marking the level from which the curve of an arch or vault rises from the upright or impost.

**STACK (or Chimney)** — Any structure or part of a structure partly or wholly exposed to the atmosphere, which contains a flue or flues for the discharge of gases.

**STRETCHER** — A masonry unit laid with its greatest dimension lengthwise in the masonry.

**STRING COURSE** — A narrow, vertically faced and slightly projecting course in an elevation, such as window-sills which are made continuous. Also, horizontal moldings running under windows, separating the walls from the plain part of the parapets, dividing towers into stories and stages, etc.

**STRINGING MORTAR** — The name applied to the method by which a bricklayer picks up sufficient mortar for a number of bricks and spreads it before laying the brick.

**SWEDGE ANCHOR** — An anchor bolt, threaded at one end and swedged or flattened in spots along the shank so as to produce greater holding power.

**TABLE (Water)** — See "Water Table."

**TAPPING** — Setting a brick down on its bed of mortar with a light blow of the trowel blade or end of handle.

**TEMPER** — To moisten and mix clay, plaster, mortar and similar materials to the proper consistency for working.

**TEMPLET** — A pattern to secure accurate and uniform shaping or spacing.

**THROAT** — The passage from the fireplace to the chimney.

**TIE** — Any unit of material used to resist the spreading of a wall, or the separation of the two solid parts of a hollow wall.

**TOOLING** — Compressing and shaping the face of a mortar joint, usually with a special tool, other than a trowel.

**TOOTHER** — A brick projecting from the end of a wall against which other wall is to be built.

**TOOTHING** — The temporary end of a wall built so that the end stretcher of every alternate course projects.

**VAULT** — An arch or a combination of arches used to span a space.

**VENEER** — A facing of masonry material attached but not bonded to the backing.

**VOUSSOIR** — One of the stones or pieces of other material used to form an arch or vault.

**WALL PLATE ANCHOR** — A machine bolt anchor, with a head at one end and threaded at the other, and fitted with plate or punched washer so as to securely engage the brickwork and hold the wall plate or other member in place.

**WALL TIE** — Strip of metal used for tying a facing veneer to the body of a wall.

**WATER TABLE** — A slight projection of the lower masonry or brickwork on the outside of a wall and slightly above the ground as a protection against water.

**WITHE or WYTHE** — Each continuous vertical section, or thickness, of masonry.

## STRUCTURAL MECHANICS

### DEFINITION OF TERMS

1. **FORCE** is that which changes, or tends to change, the state of rest or motion of the body acted upon.

2. **LOADS** imposed upon structural members may be either **concentric** or **eccentric** and they may be either **concentrated** or **uniform.**

(a) **Concentric** loads are those which are applied to the center of gravity of the cross-section of the supporting member, or more accurately, those whose resultant acts through the center of mass of the supporting member or structure.

(b) **Eccentric** loads are those applied at some distance from the center of gravity of the cross-section of the supporting member.

(c) **Concentrated loads** are those imposed on a restricted area of the supporting member.

(d) **Uniform loads** are those evenly distributed over the bearing surface of the supporting member.

Loads may be so applied to a structural member as to induce deformities — changes in volume or shape — of various types.

Members may be **stretched, compressed, sheared, twisted** or **bent.**

3. **STRESS** is an internal force that resists a change in shape or size caused by external forces. The more common stresses are:

(a) **Tension,** that stress which resists the tendency of two forces, acting away from each other to **pull apart** two adjoining planes of a body.

(b) **Compression,** that stress which resists the tendency of two forces, acting toward each other, to **push together** two adjoining planes of a body.

(c) **Shear** is that stress which resists the tendency of two equal, parallel forces, acting in opposite directions, to cause two adjoining planes of a body to **slide,** one on the other.

4. **STRESS.**

(a) **Total Stress** is the total resistance of a section of a body to deformation.

(b) **Unit Stress** is the stress on a unit of section area and is usually expressed in pounds per square inch.

(c) **Ultimate Strength** is the highest unit stress a piece of material can sustain and it is the unit stress at, or just before, rupture.

(d) **Working Stress** (allowable) is the ultimate unit stress divided by the factor of safety. The allowable stress for design purposes.

5. **FACTOR OF SAFETY.** The factor of safety for a given material is the number by which the ultimate strength must be divided to give the working stress.

6. **DEAD LOADS** are those loads on any member imposed by its own weight, and that of any other portion of the structure, of which it is a part, which is transmitted to it.

7. **LIVE LOADS** are those loads on any member which result from the occupancy or use of the structure of which it is a **part,** together with those induced by exterior forces, such as wind, snowfall, etc.

8. **TOTAL LOADS** are combined dead loads and live loads.

9. **SAFE LOADS** are loads which can be imposed upon a given member without exceeding the allowable working stress.

10. **MODULUS OF ELASTICITY** or coefficient of elasticity is the ratio of unit stress to unit deformation. The load (in lbs. per sq. in.) divided by the deformation (in inches per inch).

11. **MODULUS OF RUPTURE** or computed flexural strength is the value of the unit fiber stress S computed from the flexure formula $M = SI/c$, where a member is ruptured under transverse load.

12. **SLENDERNESS RATIO,** as used herein, is the ratio between the height of a column or wall and its least dimension.